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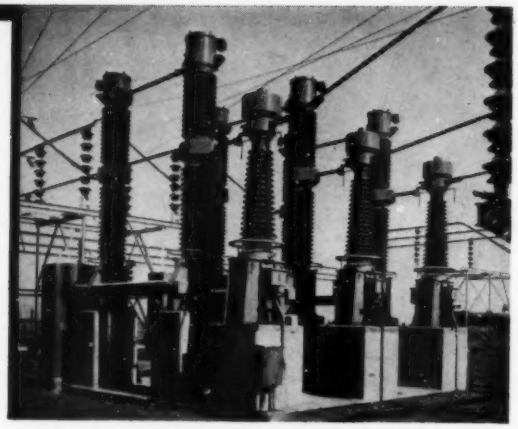

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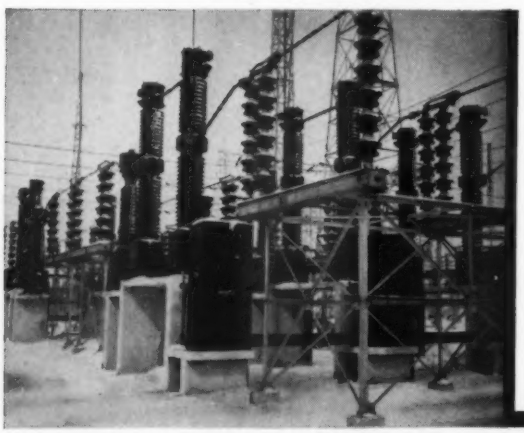

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Volume 9 Number 5

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Editor: H. STOREY, M.C., B.Sc., M.I.M.E.

PUBLISHED BY TOTHILL PRESS LIMITED
33 Tothill Street, Westminster, London, S.W.1
Telegrams: Trazette, Parl, London. Telephone: Whitehall 9233 (20 lines)

MANCHESTER: Century Insurance Building, St. Peter's Square. Telephone: Central 3101
BIRMINGHAM: 90 Hagley Road, Edgbaston. Telephone: Edgbaston 2466
NEWCASTLE-ON-TYNE: 21 Mosley Street. Telephone: 22239
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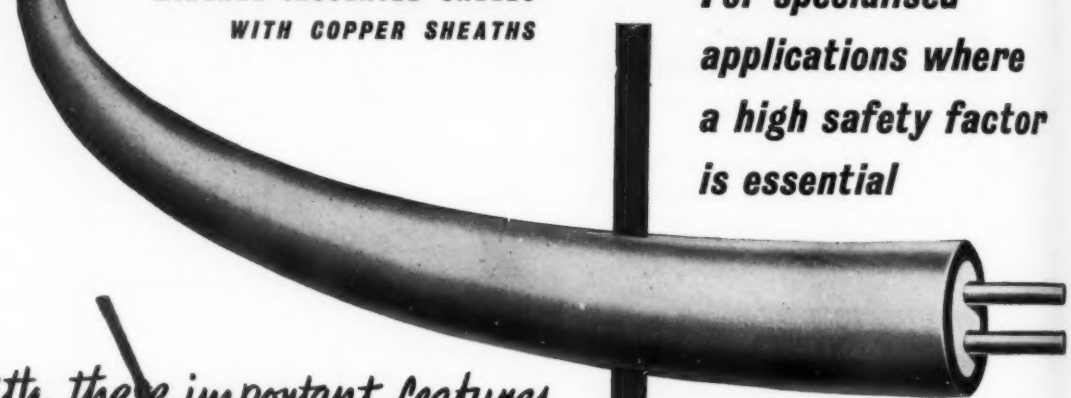

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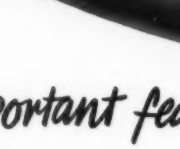

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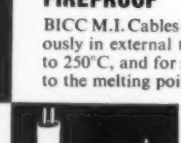

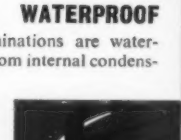

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Factors of Safety

SOONER or later, the young engineer will be told by an older and more cynical colleague that the term "factor of safety" is in fact synonymous with "factor of ignorance." It is all too true that in the case of many engineering constructions there has been a tendency in the past to play for safety by doubling the thickness of metal or concrete, and hoping that by so doing any unknown stress peaks which may have escaped the designer's thoughts will be taken care of. In the case of such structures, for example, as early railway bridges, the influence of the "factor of ignorance" is there for all to see; any competent mechanical engineer must wonder from time to time at the economic situation that allowed such enormous structures to be provided for such minor duties. The hydro-electric engineers in the early days could not be absolved from the suggestion that they, too, thought on similar lines.

In the case of the dams being constructed nowadays the hydro-electric engineer assuredly carries a greater burden of responsibility than that of most of his civil-engineering colleagues. The advent of such devices as strain gauges and delicate electrical and electronic measuring equipment has meant that many structures other than dams can be continually supervised during their working life, and moreover during the design stages model and other tests can be so arranged as to reproduce with a high degree of faithfulness the stresses likely to be encountered under any type of working condition. Thus, there is little excuse for applying the jibe that the "factor of ignorance" is still synonymous with the "factor of safety." For hydro-electric engineers, on the other hand, the establishment of a dam does not offer the relatively open conditions for supervision which apply to the majority of other structures. However much care has been taken in the preliminary surveying, there must always be some uncertainty about the settling of the ground. However great the elaboration of rock sampling, doubts must still exist as to the homogeneity of rock faces, and their behaviour under the great pressures imposed by a full reservoir behind the dam.

As mentioned in a previous leading article in this journal, there has been a wholly commendable trend, in recent years, towards the installation of elaborate networks of checking and testing devices in dams, so that engineers coming after the original designers can learn, from the lengthy accumulation of accurate data, the results of particular design efforts. Results are now coming to hand from some of the measurement projects instituted five or ten years ago, and it is remarkable to find that the civil engineers responsible for dam construction have had their original calculations, in the majority of cases, amply justified. The results we have been surveying have shown that in the majority of instances the calculated stresses come very close to those found during working, after a period of years has elapsed. Many examples could be mentioned, but to let one instance stand for many, we could mention a Swedish project, the Krokströmmen arch dam across the River Ljusnan. This dam has a curved top length of 525 ft., an upstream radius at the top of 328 ft., and a maximum height above the lowest foundation of 150 ft. The distance between the concrete thrust blocks at the two sides of the river is 473 ft. The maximum thickness of the arch is 16 ft., and it contains 30,000 yards of concrete, with 530 tons of steel reinforcements. It was built during 1949-1952, and recent figures procured by the Institution of Hydraulics at the Chalmers University of Technology of Gothenburg, have shown that the stresses, for example, in the central line of the arch are very slightly higher in the upper section, and very slightly lower in the lower section, than those calculated before the dam was begun. The figures show that the measured stress was 26 kg. per sq. cm., whereas the calculated figure was 22 kg. per sq. cm. In the lower part the measured figure was 19 kg. per sq. cm., against a calculated figure which was also 22 kg. per sq. cm.

The close correspondence of these figures is remarkable; but it is matched by many similar observa-

tions we have noted from various parts of the world. The inference to be drawn is obviously that hydro-electric engineers, faced with the very considerable responsibility of designing a dam, apply only the right and proper "factors of safety," and not the "factors of ignorance" used when designers are either ignorant or over-cautious in regard to possible increased stresses during the working period.

In the other aspects of hydro-electric design—the manufacture of pipelines, penstocks, turbines and generators—it has been possible, as with other structures, to apply supervisory equipment to enable a continuous check to be kept of the stressing of every part of the plant. Figures from these sources have shown that in the past strength has sometimes been applied in the wrong places, and it has been revealed that certain parts, for example under water-hammer conditions, tend to be over-stressed. Comparisons of detailed drawings of more recent installations with those of only ten or fifteen years ago, show that lessons have been learned.

The pressure on hydro-electric engineers to develop their resources with the utmost economy has meant that any overdesigning, through the application of ignorance factors instead of safety factors is even more to be deprecated than in the past.

On the whole, the water-power industry has utilised its raw materials—steel, concrete, labour, and skill—with due care and without the prodigality born of ignorance or cowardly over-caution. Thus, whereas the designers of many other engineering constructions have had to revolutionise their ideas in recent years in order to bring down costs, those concerned with hydro-electric projects have had to do little more than to accelerate a process which in any case has been operating continuously.

Sixth Congress on Large Dams

FURTHER to the note in our April issue giving details of the technical programme of this Congress, it has now been announced that the Congress will be held from September 15 to 20, 1958, in New York. Papers must reach the Central Office of the International Commission on Large Dams by January 1, 1958. The British National Committee has asked authors to submit the titles, scope and approximate lengths of their proposed papers not later than June 30, 1957, and doubtless generally similar arrangements will obtain in other countries.

World Power Conference Belgrade Meeting

THE general programme of the Jugoslav Sectional Meeting of the World Power Conference, to be held in Belgrade from June 5 to 11, 1957, has now been published. The main theme of the meeting will be "Power as a Factor of Development of Underdeveloped Countries." The papers will be divided into two main divisions; Economic Aspects, and Technical Aspects, and the first section in the technical division will be devoted to the integral utilisation of water resources.

During the conference a whole-day excursion will be paid to the Zuornik 92 MW plant on the Drina River and post-conference tours will include visits to the following hydro-electric installations: Vlasina in Southern Serbia, Mavrovo in Macedonia, Jablanica in Bosna, Hercegovina south of Sarajevo, Novi Vinodol in Dalmatia, Jajce in Bosna, and Moste in Slovenia.

Conference on Turbines and Pumps

A FIVE-DAY congress on hydraulic turbines and pumps is being organised by Société Hydrotechnique de France from June 23 to 28, 1958. The actual conference will take place at Aix-en-Provence from June 26 to 28, and will cover seven subjects: Development of turbine technology and its influence on civil engineering; turbine governors; turbine efficiency and cavitation problems; turbine applications; pump-turbines; pumps; borehole-drilling turbines. The days preceding the conference will be devoted to visits to the works of turbine manufacturers and to hydro-electric installations.

The Kariba Flood

SEEING that the flooding on the Zambesi last March made newspaper headlines, a brief factual account of the situation at the Kariba dam site may not be out of place. A rise in the upstream levels of the Zambesi and its tributaries gave nearly a fortnight's warning to the engineers at the site, and all vulnerable equipment was removed before the flooding took place. This occurred at 11.30 p.m. on March 14, when the cofferdam, which has a crest level of 1,303 ft., was overtopped. The river rose rapidly at first and then became steady on the 22nd at a level of 1,316·3 ft.

The cofferdam crest level corresponds to a flow of 210,000 cusecs, which, on the basis of the comparatively scarce records available, represents a flood of about a four-year frequency. As is common practice in cofferdam design, the crest level was fixed on a basis of a legitimate economic risk of the occurrence of a flood in excess of this amount, and in fact, the cofferdam was deliberately designed for overtopping. The actual flood is thought to have been of the order of 280,000 cusecs, although this is only an estimate, and floods exceeding this amount have been known to occur. From an engineering standpoint, therefore, the flooding at Kariba was neither unprecedented, unforeseen, nor unprepared for.

Shawinigan Report

ACCORDING to the annual report for 1956 of The Shawinigan Water and Power Company, the total amount of electricity sold during the year was 9,898,421,250 kWh, an increase of 7·2% over 1955. The primary power peak load on the system rose to 1,600,800 kW from 1,500,000 kW. in 1955. It was necessary during the year to operate all generating facilities at their maximum capacity without the normal margin of reserve.

Reflecting the increasing electrical requirements in the territory served by Shawinigan, the Company's capital expenditures for new construction during the year totalled $23,767,000 compared with $14,994,000 the previous year. Of this amount, the largest item was $9,680,000 for construction work at the Beaumont hydro-electric development, Shawinigan's seventh on the St. Maurice River. Construction of the

new 246 MW generating station was started in the spring and the first of its six units is scheduled to go into service later in 1958 with full operation early the following year. Total cost of the new plant together with allied installations is expected to be about $56,000,000. Other major items of capital expenditure in 1956 were for new substations and improvements and additions to the Company's distribution system; for completion of new terminal stations at Chaudiere and Kingsey station and a linking 230 kV line, all of which are part of the Company's programme to expand transmission facilities south of the St. Lawrence; and for expansion of Quebec terminal station.

Cook Strait Cable Discussions

IN the face of strong opposition from Ministry of Works representatives, the joint committee planning New Zealand's future power needs is understood to be recommending an early start with the Cook Strait cable, capable of transmitting power as required between the two islands. This power would come from the planned new hydro-electric works at Benmore. It is understood the cable would be laid in two years, but because Benmore would take longer to build and put into operation, 1965 is mentioned as a possible date for operating the cable.

It is believed that the Ministry of Works representatives wanted the Cook Strait cable shelved in favour of the Wairakei geothermal project. Power supply and State Hydro-electric Department representatives are said to have favoured the Cook Strait cable, because they believe the Wairakei project's capacity still has to be proved. According to one source there was a stage when the rift threatened to bring the deliberations of the committee to a stop. The committee's views are gathered in a report which will be considered at a further meeting. The committee has power to make recommendations only. Decisions are a matter for the Government.

The 69,000 kW geothermal plant is now being built at Wairakei. The Ministry of Works, it is believed, thinks the ultimate capacity should be treble that amount.

Electricity Supply in Europe

THE OEEC has published a report collating the replies to the Seventh Enquiry of the Electricity Committee regarding achievements and forecasts for the period 1954-61 for the European electricity supply industry. During the period 1953-56 the average annual increase in consumption was 9%, this being some 2% higher than that noted in previous years. Output from public-utility thermal plant over this period increased slightly faster than hydro output, the former showing an average annual increase of 10·2% as against only 7% for the latter. This tendency was particularly marked in 1954-5, when the increase in thermal output was 12·2% as against 4·2% for hydro.

Estimates for future installed capacity and for production are definitely higher than those appearing in the two previous enquiries. Demand continues to increase faster than estimated, and the various countries are obliged to readjust their programmes each year to allow for the general trend of economic development. Total production for 1955 estimated at the time of the Fifth Enquiry was 342,700 GWh, whereas the actual production as reported in the Seventh Enquiry was 358,600 GWh (a difference of 6%). The estimate for 1958 was 398,800 GWh and it is now amended to 446,300 GWh. For the period 1956-61 the average annual increase in thermal production is estimated to be 8·5% as compared with 6·5% for hydro production, representing an overall increase of 7·5%. By 1961 hydro production is likely to represent only 36% of the total, as against 41% in 1950.

Power exchange between interconnected Member countries has continued to increase each year. Exports increased from 5,096 GWh in 1953 to 6,546 GWh in 1955, i.e. 14% per year. The heaviest exchange took place in the case of Germany, where exports for this period increased by 35%. Exports from Sweden are expected to rise from 188 GWh in 1954 to 500 GWh by 1959.

British Hydromechanics Report

THE Annual Report, to September 1956, of the British Hydromechanics Research Association discloses that the organisation has now become so well established that it is able to make some reduction in the scale of subscriptions.

The Association has extended its facilities in the laboratory for investigating the design of hydraulic structures for hydro-electric, irrigation and water-storage schemes. These include a 22 in. wide flume 32 ft. long by 5 ft. deep in which a discharge of up to 7½ cusecs may be attained, which has been used for testing sectional models of spillway structures, and a 20 ft. square watertight basin in which small-scale rigid-boundary models of complete spillway dams and channels, forebays for hydro-electric schemes, etc., have been tested. During the year work has been completed on models of the spillway structures for the Roseires dam irrigation scheme on the Blue Nile, and work is in progress on models of the intake structures for the Kariba hydro-electric scheme on the Zambesi, and a model of the spillway channel for Daer reservoir, a water-storage scheme in Lanarkshire.

The use of air as a working fluid for hydraulic experiments has been developed and has been applied to a large model study of the flow in the intake works and penstocks of the Kariba power station, a fundamental study of the hydraulic effects of surface roughness, and observations of the flow in a centrifugal pump casing.

Important and successful work has been carried out on the measurement of flow within centrifugal pumps, and the researches on the design of suction intakes, on seals and glands, and on the hydraulic transport of solids are continuing.

Uganda Electricity Developments

THE extension programme of the Uganda Electricity Board for the years 1956-1960 was passed by the Legislative Council of Uganda on 14th March, and the borrowing powers of the Board were raised from £27 million to £33 million.

This programme forms part of Uganda's Capital Development Plan up to the year 1960.

Excluding any expenditure on a second dam which the Board are now considering, the Board expect to

spend about £6 million on main transmission, distribution fed by hydro-electric power from the Owen Falls scheme, and on self-contained schemes for up-country stations, which are too remote to be supplied from Owen Falls. The Board estimates that after 1957, coinciding with the start of the bulk supply to Kenya, the trading account will begin to show a surplus, after allowing for operating, maintenance and management costs and providing in full for sinking fund and loan interest charges, except on capital assets not in commercial use, and after assuming that the full programme is carried out. The proposed programme of development will therefore be self-supporting.

Electricity Prospects in Victoria

THE State Electricity Commission of Victoria, Australia, according to its Thirty-Seventh Annual Report to June 30, 1956, has engaged Ebasco Ltd., New York, to examine and report on the forecasts of future loading, the plans for meeting the future electricity needs, and the capital expenditure necessary to carry out these plans. An interim report from Ebasco Ltd. estimates that the maximum demand will rise from 943 MW in 1956 to 1,840 MW in 1965, and that an expendtiure of between £282 and £313 million will be required during the nine years.

Most of the projected increase of capacity will be from thermal stations, but by 1965 the State should have at its disposal about 200 MW of low-load-factor power from Snowy Mountains, out of a total of 920 MW in 1982 when the scheme is completed. The load factor of Snowy's T1 and T2 stations is stated to be 30%, and the load factor for the completed scheme 24%. The value of the Snowy scheme will lie in its operating in conjunction with thermal base-load stations of about 80% load factor to satisfy a total system load factor of 55-60%. Estimates of total costs show a slight saving to Victoria in using Snowy power as compared with setting up low-load-factor thermal stations of equivalent capacity. Negotiations between the Commonwealth, New South Wales and Victoria have marked a stage at which the detailed terms of participation in the Snowy scheme are generally acceptable to the three parties.

During the year covered by the report, 35·5 MW was added to the State's installed capacity, of which 30·8 MW was at Kiewa. The 120 MW station at Eildon, described in our April issue, is, of course, a subsequent addition.

Progress in British Columbia

FROM the technical standpoint the highlight of the annual report for 1956 of British Columbia Power Corporation Limited is the recording of the completion of the Vancouver Island cable. Details of this achievement have already been given in past issues of this journal, and it suffices to repeat that a 120 MW 132 kV link has been completed by means of five B.I.C.C. submarine cables traversing 19½ miles in two sections, the longest underwater crossing requiring unspliced cable lengths of 17 miles.

Work continued during the year on the Cheakamus development, and by the end of the year 5½ miles had been completed of the 6¾ mile tunnel to carry water from the Cheakamus River to the generating station on the Squamish River. This station, which should be completed this year, will contain two 70 MW sets and, it is believed, will be the world's largest remote-controlled hydro-electric station. Also coming into operation in 1957 will be a 22 MW unit at La Joie dam and a 30 MW unit at Clowham. Work continues on the second stage at Bridge River, which will contribute another 268 MW of capacity. Reference is made to the Powell River 132 kV line and to developments in irrigation. Important developments are also recorded in natural gas.

Strathfarrar and Kilmorack Project

THE North of Scotland Hydro-Electric Board have published their Strathfarrar and Kilmorack scheme (Constructional Scheme No. 30) which is intended to harness the waters of the Rivers Farrar and Beauly for the production of 268 million kWh annually.

The catchment area which it is proposed to ultilise extends to 350 sq. miles in the Counties of Inverness and Ross and Cromarty. The flow from this area will be put to use in four power stations with a total installed capacity of 102 MW. Two of these will be in the Kilmorack Gorge and two in Glen Strathfarrar. The Kilmorack stations will use the flow not only from the two stations in the glen but also from the Fasnakyle power station, the regulated discharge of which is controlled by Loch Mullardoch and Loch Benevean, the reservoirs of the Affric scheme. At present prices the estimated cost of the scheme is £14,250,000. The construction of the works, which will take about five years, will employ a peak labour force of between 750-1,000 workers.

Power Production in Norway

THE production of power in Norway will increase on an average by at least 2,000 million kWh per annum up to 1960, according to a report presented to the Norwegian Storting by the Ministry of Industrial Affairs. By 1960 production will have reached altogether 32,000 million kWh. The report also shows that the amount of power going to civilian uses is steadily increasing.

Mr. Leslie Ward

WE regret to announce the death on March 13, at the age of 63, of Mr. Leslie Ward, Manager of the Manchester office of Tothill Press Limited, the company which owns and publishes this journal. Mr. Ward had been associated with the company for 28 years. He joined Tothill Press in 1929 and confined himself chiefly to the advertising side of the business. His work brought him into touch with some of the most prominent manufacturing concerns in the country, and in 1940 he assumed control of the Manchester office. Mr. Ward lived to see the business in Manchester, where he and his staff were responsible for the representation of twenty publications in one of the most important industrial areas of the country, grow out of all proportion to its early beginnings. He had endearing personal qualities and was held in high regard by all with whom he came in contact.

Mr. Ward has been succeeded by Mr. H. R. Bowker.

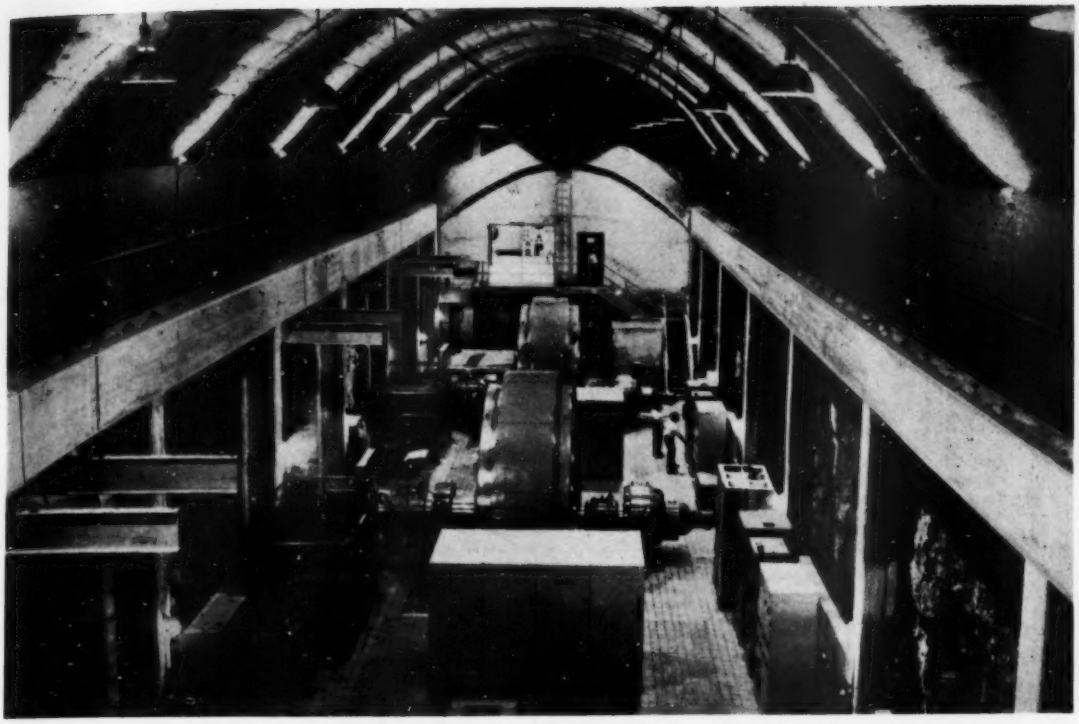

Fig. 1. Interior view of Guayabo power house

Guayabo Hydro-Electric Project

A description of a hydro development in Central America featuring an underground power station of unusual design in which the horizontal shaft setting of the generating units, in conjunction with horizontal draft tubes, resulted in economical construction and high hydraulic efficiency

By RICHARD D. HARZA and THADDEUS ZOWSKI*

THE Guayabo hydro-electric project on the Rio Lempa in El Salvador is now approaching completion of its first stage of development. Two 15 MW sets are operating and a third set of the same size will commence operation shortly. The power will go to fill the rapidly growing needs of the smallest but most densely populated of the American republics. The project is made up of the following principal elements: the main concrete dam (officially called "La Presa 5 de Noviembre"); the resulting reservoir in the Lempa; a small overflow dyke which will also act as a "fuse plug" type supplementary spillway; the underground powerhouse; the transmission system; and the Lake Guija control works.

This project includes a number of features of special interest. First, the Lempa River is the largest American river flowing to the Pacific south of the Sacramento in California, and the Guayabo Project is the first development on it. Second, the Guayabo Project is believed to be the largest and most modern hydro-electric development in the Central American area. Third, this was the first project of any type financed by the International Bank for Reconstruction and Development (World Bank) in Central America. The financing represented a considerable problem because of the large size of the project and the small size of the country (the population of El Salvador is 2 million). At the time of financing, the cost of the project equalled the annual budget. Fourth, the Guayabo project introduces a new system of underground power station design involving the use of horizontal shafts joining the alternators with the turbines located *beside* rather than *under* the generator hall (see

* Respectively Vice-President and Chief Mechanical Engineer of the Harza Engineering Company, Consulting Engineers, Chicago.

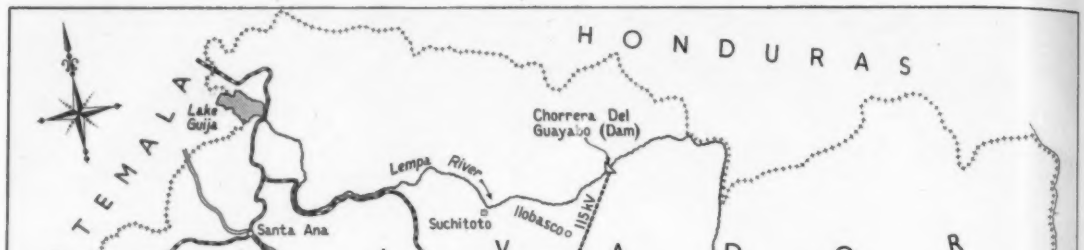

Fig. 2. Map showing location of Lake Guija, Guayabo power station and transmission system

WATER POWER, March 1951). The objectives of this design, namely to reduce power station construction costs and to increase hydraulic efficiency, have both been attained.

Project Location

Referring to Fig. 2, it will be observed that the project is centrally located in El Salvador at the Chorrera del Guayabo rapids on the Rio Lempa, and that the transmission system extends to the two most important load centres of the country, San Salvador and San Miguel.

The control works on Lake Guija are located on the natural discharge channel of the lake, which is situated on the north-western border between El Salvador and Guatemala. The lake is expected to provide a usable storage of 351 million cu. m. during the dry season each year.

The unregulated low flow of the Lempa at the Guayabo site is in the range of 18·4 to 20 cu. m. per sec., but with the regulation achieved by the dam and the Guija control works this will be raised to 50 cu. m. per sec. Further hydrologic information is included in Table I.

Normal annual floods at Guayabo are of the order of 4,000 cu. m. per sec. However, a record occurring in 1934 in unusual circumstances, when a hurricane from the West Indies crossed the isthmus divide and entered the Lempa catchment, resulting in a discharge estimated at 15,000 cu. m. per sec. There is no other record of a hurricane in the area.

TABLE I—SIGNIFICANT DATA

Stream Flow	
Drainage area above dam	9·785 sq. km.
Maximum recorded flow	15,000 cu. m. per sec.
Design flood	18,500 cu. m. per sec.
Minimum recorded flow	18·4 cu. m. per sec.
Minimum regulated flow (dry year)	43 cu. m. per sec.
Mean annual flow	230 cu. m. per sec.
Reservoir and Dam	
Reservoir area, approximately	20 sq. km.
Normal operating level, above mean sea level	180 m.
Maximum drawdown	13 m.
Total reservoir volume	324 million cu. m.
Net usable storage	156 million cu. m.
Length of dam	455 m.
Length of gated spillway	100 m.
Volume of concrete in dam	250,000 cu. m.
Plant Operating Date	
Rated net head on turbines	50 m.
Operating net head range	41 to 56 m.
Installed capacity	45,000 kW.
Ultimate installed capacity	75,000 kW.
Estimated average annual output	170 million kWh.

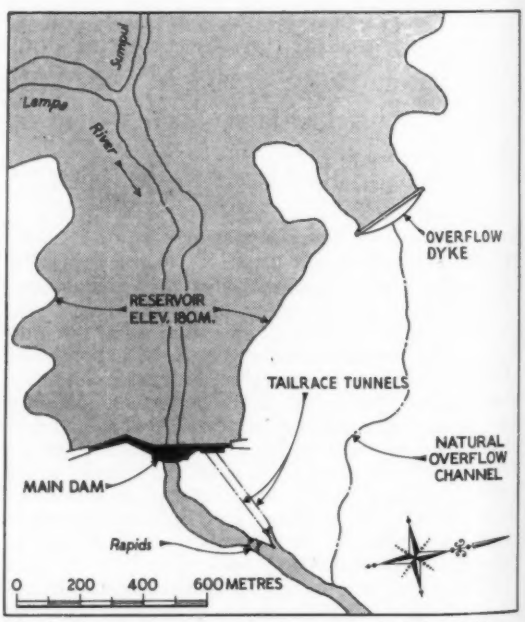

Fig. 3. Sketch plan showing dam, storage lake, tailrace tunnel and overflow dyke

Earthquake and volcanic eruptions are known in El Salvador. There are no volcanos in the immediate region of Guayabo, but full cognizance of the possibilities of earthquake was taken in designing each element of the project.

Engineering Studies

To achieve the most suitable design for the Guayabo project, a complete set of engineering studies and tests was undertaken. An enumeration of these studies and tests, together with certain descriptive comments follows.

A programme of stream gauging supplemented by a study of channel topography and previous high-water marks was undertaken at the site. Current meter readings were made from a cable-way. At the same time, specimens of the sediment-carrying water were taken at various depths and river discharges. Based on data thus obtained, it was estimated that the Guayabo reservoir will have a useful life of from 40 to 66 years.

The foundation was diamond drilled, and the final location of the main structures and the extent of the grouting programme were based on the results of these borings. The length of holes drilled was 820 m.

To learn more accurately the condition of the rock in the area of the proposed power chamber, a test drift 2 x 1·5 m. was driven about 2·5 m. above the proposed arch for a length of 360 m. Later this drift was utilised as a drainage and grouting gallery during construction, and subsequently as a ventilation and emergency exit gallery for the completed power station.

River water was chemically tested to determine its suitability for use in concrete and its corrosive effect on metals, and tests were made of the material available locally for concrete aggregate. The "mortar bar" series of tests and the petrographic analyses revealed that this aggregate material produced a slight alkali reaction with cement. An addition of pozzolanic material (fly ash) was made to correct this situation.

Engineering studies of alternative types of structures were made for each main element of the project.

Concrete-arch, multiple-arch with massive buttress, and concrete gravity-type dams were studied. The conventional above-ground setting, an underground setting with a vertical-shaft arrangement, and an underground horizontal-shaft type of power sta-

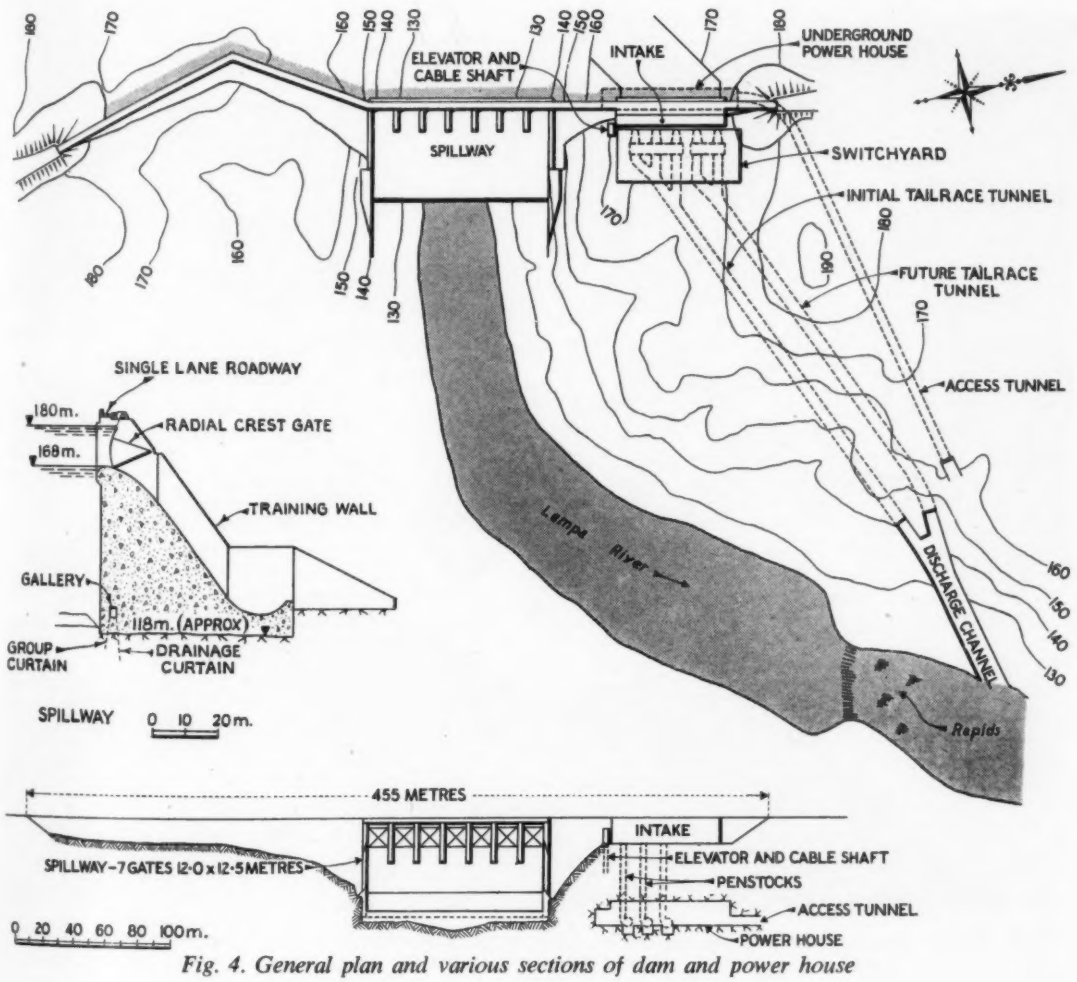

Fig. 4. General plan and various sections of dam and power house

tions were studied. Laboratory tests on a reduced-scale model of the spillway were conducted to determine spillway capacity, the effect of flood discharge on foundation, and the effects of flood discharges on tailwater. A comparison of the action of the spillway bucket in the model and (later) in the prototype may be seen in Figs. 5 and 6.

A study of the market for power was made. Three avenues of approach were taken in this study; an analysis of existing and prospective individual and industrial consumers, a study of the growth of power and energy demand in other similar countries, and a comparison with generally accepted prediction curves of load growth. Current growth bears out quite well the findings of these studies, and the growth of demand in 1955 amounted to 11%.

Sequence of Events

In tracing the development of the project, the following dates are significant. In May, 1947, the consulting engineer was retained to prepare a feasibility report. In September, 1948, the National Assembly of El Salvador enacted a law empowering its autonomous agency, Comision Ejecutiva Hidroelectricadel Rio Lempa (CEL), to issue bonds and borrow money to finance the work. On December 14, 1949, the World Bank loan was approved. In June, 1950, the general construction contract was signed after the adjudication of international bidding.

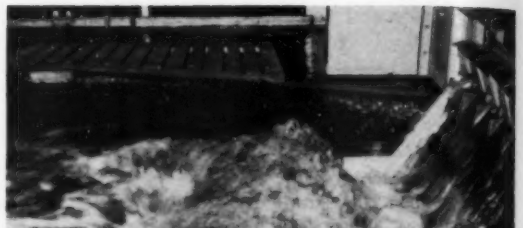

Fig. 5. Scale model of spillway showing flood discharge and standing wave. Model tests directed by Dr. Lorenz Straub at St. Anthony Falls Laboratory

Fig. 6. Aerial view of Guayabo dam during flood discharge; note the similarity of flow conditions between model prototype (Fig. 5)

The first generating unit started to deliver power in June, 1954.

The rather long period between the start of engineering work and the first delivery of power, was largely due to the fact that the project was of unprecedented size for the area, and that the legal requirements in connection with the financing were time consuming.

Dam and Spillway

The dam consists of five important parts; the right-bank wing dam, the spillway section in the main river channel, the short left-bank wing section, the intake structure, and finally the overflow dyke "fuse plug." The first four parts of the dam are concrete founded on an agglomerate rock with basalt intrusions, the fifth being founded on alluvial sands and clays. The dam is of gravity section, and has a grouting and drainage gallery located near its upstream heel. The right-bank wing section was dog-legged in plan to avoid an undesirable foundation condition.

The main spillway section contains seven radial gates 12·5 m. wide by 12·0 m. high. These gates are electrically actuated, and can be controlled from the spillway deck or from the power station. Spillway capacity at normal pool is 7,000 cu. m. per sec., and a 3-metre surcharge on the pool increases this to 10,000 cu. m. per sec. Should the pool be raised even further by an extreme flood, the "fuse plug" earth dam on the left bank would wash out, adding 8,500 cu. m. per sec. more capacity. The channel of this fuse plug section is high enough so that the power pool could be maintained at generating level even during the rebuilding of the fuse plug.

Underground Power Station

The selection of the underground type of power station was dictated largely by the high tailwater rise conditions in the narrow river canyon and the greater freedom afforded by this type of construction in locating the main structures in the most advantageous positions. The freedom from restrictions of surface topography permitted the power station to be located directly below the intake, making the penstocks as short as possible. The power station was placed low enough to discharge the water from the turbines through a tailrace tunnel into the river at a point downstream of the rapids, thus utilising the head on the rapids in addition to the head created by the dam. To develop the same total head with an above-ground powerhouse would have required either a larger dam located downstream of the rapids, or a dam upstream connected to a powerhouse below the rapids by a long pressure tunnel or conduit with a surge chamber, followed by individual penstocks equipped with shut-off valves at the turbine inlets. Either of these alternatives would have cost more than the underground scheme. In both cases the surface powerhouses would have required heavy and expensive walls to withstand the tailwater rise of more than 20 m. during maximum floods and about 6 m. during annual floods. In an underground station excavated low enough in rock to assure satisfactory turbine setting in relation to minimum operating tailwater level, the turbine head cover is the only important part in the power station exposed directly to tailwater pressures. Design of the head cover to withstand maximum tailwater pressure does not present any difficulty.

In addition to a substantial saving in concrete structural work, the underground station offered the advantages of considerable excavation work, on which local unskilled labour could be employed, and of requiring a minimum of imported materials. Also the construction time was shortened due to the lesser amount of concrete work and to the greater freedom from weather hazards, such as tropical rains and floods. The underground location is also safer from rock falls, landslides, or aerial bombardment.

The estimated total saving in construction cost of the horizontally arranged underground scheme over an above-ground scheme was about half a million U.S. dollars, or about 2·5 per cent. of the total project cost. In addition, the annual maintenance costs of an underground station are substantially lower due to elimination of buildings, exposed penstocks, surge tanks and other outdoor structures.

Intake and Pressure Conduits

The power station intake structure is integral with the left-bank section of the dam. With the turbine located 41·2 m. directly below the intake sill, a very desirable layout was obtained, using vertical penstocks which lead the flow directly into the spiral cases. This not only minimised the expenditure on pressure conduits, but also assured smaller pressure variations under load changes, minimum hydraulic losses in the penstock, and excellent inflow conditions at the turbine inlet. A sectional arrangement of the intake structure, penstock, and power station is shown in Fig. 7.

The intake structure was built to accommodate the ultimate installation of five main turbines. The intakes are protected by vertical trashracks designed for a differential pressure head of 3 m. and a normal flow velocity of 0·75 m. per sec. through their gross area. Clear opening between rack bars is approximately 90 mm.

The water passages are controlled by intake gates 4·5 m. wide by 5·0 m. high. Gates of the submerged radial type are used rather than vertical lift type, because of their reliability in closing by gravity against full flow, and their ease of opening against a full unbalanced head. Each gate is operated by an 18-ton capacity electric-driven, cable-drum type hoist, located on the downstream side of the intake deck. As the intake gates at Guayabo are the only shut-off device ahead of the turbines, it was considered essential to provide gates of a type which would be most certain to close under any flow condition, including maximum turbine discharge under runaway speed conditions that might occur if the governor should fail to close the turbine wicket gates upon loss of generator load. The radial gate can always be closed in an emergency by releasing the hoist brake and allowing the gate to descend under its own weight at a speed which is controlled by a centrifugal brake on the hoist motor. The hoist brake is released by unlatching a weighted lever by means of a direct-current solenoid operating on the station storage battery system and actuated by a pushbutton in the station control room. Normally, the gate hoists are operated from the control room or from auxiliary control stations at each hoist. Gate position is indicated electrically in the control room.

The intake gates are of welded-steel construction and have moulded rubber seals along all four edges.

The gates can readily be opened against full head-water pressure without using by-pass valves to equalise pressures before lifting. They are normally lifted a small amount first to fill the penstock, then fully raised.

The penstocks have an internal diameter of 3,500 mm. and consist of vertical shafts driven through solid rock and subsequently lined with concrete and steel. The welded liner is made of plate steel $\frac{3}{8}$ in. thick to prevent excessive deformation during fabrication, handling and installation, and is somewhat thicker than required by stress considerations alone.

Horizontal Shaft Setting of Turbines and Generators

The horizontal shaft setting was intended first to reduce construction cost in terms of power chamber excavation, substructure concrete, and assembly time. Since the width and height of the power chamber can be considerably reduced with the horizontal settings and the spacing of units is no greater than with the vertical shaft setting, very considerable savings in excavation were realised. A recent comparison of this design with those of nine other modern vertical-shaft type underground power developments, using the generator diameter as the critical parameter, indicates that the Guayabo station requires only 60% as much excavation as the average of the others. The savings in substructure concrete appear to be at least as great. Reduction in construction time resulted from the reduced work quantities and also from the fact that, with the horizontal setting, both the generator and the turbine can be assembled at the same time. From an operational standpoint the remarkable freedom from noise and vibration at Guayabo is important; this is attributed to the turbine being more truly mounted in solid rock, and to the entire unit being supported on bearings which are more closely connected to solid rock, than is the case in a vertical-shaft arrangement. A final saving in cost comes from substituting the horizontal straight-axis conical-type draft tube for the more complicated elbow type used with a vertical setting.

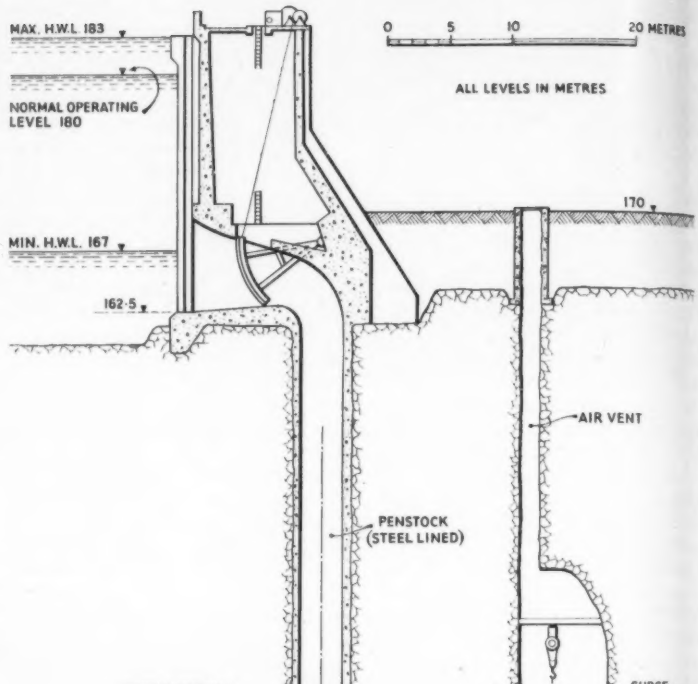

Fig. 7. Section through intake structure, penstock and power station

The second objective of the horizontal-shaft arrangement was to obtain greater hydraulic efficiency by eliminating the bend which usually occurs ahead of the spiral case in a vertical-shaft setting and eliminating the elbow bend in the draft tube. After the plant started operation, tests were conducted which revealed a marked increase of efficiency over that anticipated, as described later in this article.

The horizontal setting has the following disadvantages which must be taken into account:

(a) The turbines are not within reach of the main power station crane, therefore special handling devices must be provided in the turbine vaults. In the Guayabo station two hand-operated monorail hoists are provided for this purpose.

(b) The horizontal setting is not favourable to the frequent use of the generating units as synchronous condensers, for improving power factor and voltage regulation. The horizontal draft tube, with the runner below the tailwater level to assure adequate submergence of the outlet, does not permit ready evacuation of the water around the runner by admission of compressed air as is done in vertical turbines. To permit the runner to rotate in air, the draft-tube gate would have to be closed and the water pumped out.

Careful consideration of these disadvantages was given during the design stage, but it was concluded in the design phase, and has since been confirmed during the constructional and operating phases, that the disadvantages are greatly outweighed by the advantages. Two other underground stations of similar design

and by the same consulting engineer are now under construction.*

Turbines

The main turbines are of the horizontal-shaft single overhung-runner Francis type, mounted in welded-steel spiral cases. The cases are embedded in concrete in individual turbine vaults excavated from rock on the downstream side of the power station. The turbines discharge into conical draft tubes which are steel-lined and embedded in concrete. Each turbine has a pedestal bearing for supporting the runner and shaft at the head cover. The turbine shaft is connected through an intermediate section to the generator shaft. All removable turbine parts can be taken out to the generator hall after removing the intermediate shaft section, without dismantling the generator. Handling of the turbine parts is aided by two 12-ton overhead monorail hoists which travel on rails in the turbine vaults between the turbine and generator hall. The rails extend into the generator hall far enough to allow placing of the turbine parts within reach of the main crane.

The turbines are designed to have a rated capacity of 21,000 m. horsepower each at full-gate opening when operating at 200 r.p.m. under a net head of 50 m., to match the rated generator capacity of 15,000 kW. The discharge from each turbine is approximately 35 cu. m. per sec. The turbines are designed to operate with good efficiency at net heads between 41 and 56 m. Under the maximum operating head of 56 m., the calculated maximum hydraulic thrust on the generator thrust bearing is 80,000 kg. at 200 r.p.m. and 45,000 kg. at the maximum runaway speed of 400 r.p.m.

The runner is of cast steel, with 15 buckets cast integral with the band and crown, and has a discharge diameter of 2,330 mm. Replaceable wearing rings are provided on the runner periphery and on adjacent stationary parts where there are small running clearances. The throat sections of the distributor adjacent to the wicket gates are protected with replaceable facing plates of carbon steel. A replaceable throat liner in the discharge ring forms a short cylindrical section between the runner and the draft tube.

* The Ambuklao station in the Philippines and Maithon station in India.

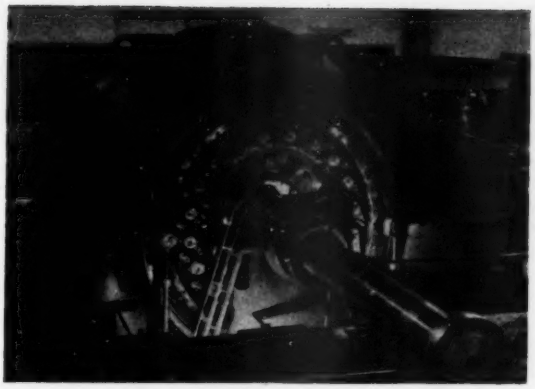

Fig. 8. Turbine assembly in manufacturer's works (Theodor Bell & Co. Ltd., Switzerland)

The turbine shaft is of 520 mm. nominal diameter, enlarged to 570 mm. at the bearing. The main shaft seal in the head cover has two sets of spring-loaded carbon rings which seal against the faces of two collars mounted on the shaft. Labyrinth seals are also provided between the runner crown and the head cover. The main bearing is ring oiled and has a horizontally split bearing shell with passages in the lower half for circulating cooling water. A starting oil pump is also provided. The bearing pedestal is bolted to the centre of a heavy cast-iron foundation beam at the ends of which are mounted the wicket-gate servomotors.

A cast-steel stay ring with 10 vanes guides the water into the runner. It was made in four sections which were welded to the spiral case plates at the manufacturers' shop. The spiral case has an inlet diameter of 2,700 mm. and is made in four radial sections with machined flanges for bolting the sections together at the site.

The cast-steel head cover is sectionalised concentrically into an inner and outer cover to facilitate shipment and to permit removal of the runner without disturbing the wicket gates. The inner cover is made in halves and supports the main shaft seal. The outer cover is made in quarters and contains the gate stem bushings and seals. A head-cover drain, connected

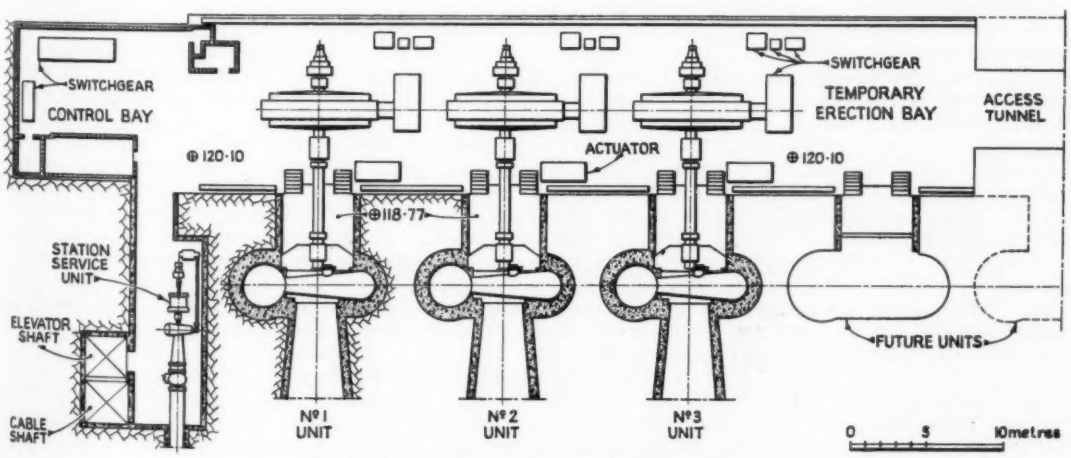

Fig. 9. Part sectional plan of power station

with the draft tube by a 150 mm. pipe around the spiral case, relieves water pressure on the inner cover and reduces hydraulic thrust on the runner. The head cover is designed to withstand safely a maximum pressure equivalent to 32 m. of water, which may occur in the draft tube due to a surge in the tailrace tunnel under flood conditions.

Flow through the runner is controlled by 20 wicket gates, of welded construction, and an automatic Helios high-pressure centralised lubrication system supplies grease to all bearings of the gate mechanism and to the main-shaft seal. The shop assembly of one of the main turbines is shown in Fig. 8.

Each turbine has an automatic oil-pressure operated speed governing system of 8,000 kgm. capacity, sufficient to open or close the wicket gates completely in 2·5 sec., with the turbine operating under the maximum head. The governors were built by the turbine manufacturer to meet contract specifications and are of the cabinet acuator type. The principal elements, including the actuator, oil-pressure receiver, pumping unit and sump are enclosed in a neat metal cabinet, with the indicating instruments and controls mounted on the front panel. The speed-responsive governor head is operated from a small permanent-magnet generator connected to the main generator shaft, so that its frequency is always proportional to turbine speed. This frequency responsive element is of Brown Boveri manufacture and has a static frequency measuring system; it is in no sense a mechanical speed-measuring device consisting of a centrifugal pendulum with electric motor driven flyballs. It has two drums mounted on a shaft and acted upon by rotating fields of two separate wound stators. The current in one stator is capacitive owing to the use of a condenser in series with the winding, while the current in the other is inductive. The two rotating fields work in opposition to each other so that when the frequency is normal the torques acting on the drums balance each other and the shaft remains at rest. When the frequency is above or below normal, one field overcomes the other and rotates the drum and shaft, which in turn operates the control valve of the hydraulic system regulating the turbine servomotors. This type of governor head has been used successfully in some European and other hydro stations, and its regulation of the Guayabo turbines is considered satisfactory.

Generators

The main generators have a rating of 16,667 kVA each at 0·90 power factor, 13,800 V, three phase, 60 cycles, 200 r.p.m. They have horizontal shafts with two pedestal-type journal bearings, the one at the turbine end being combined with a thrust bearing. The thrust bearing is designed for a maximum axial load of 80,000kg. from the turbine runner, acting toward the turbine, and a reverse thrust of 22,500 kg. Oil is circulated by a motor-driven pumping set run off an a.c. supply, supplemented by a standby set driven by d.c. The generators are built with solid rotor poles and without damper windings. The exciter, pilot exciter, and permanent-magnet generator for the governor are direct connected to the generator shaft. The rotating parts have a flywheel effect of 850,000 kg-m^2. Generator brakes with two cylinders operated by oil pressure from a hand pump are used to force a set of brake shoes against a brake ring mounted on the rotor, thereby stopping rotation in the minimum time.

The generators have closed-circuit cooling, four air coolers being in service to maintain in-going air temperature below 40° C under normal load. The generator construction permits stator shift in the direction of the turbine to facilitate access to the stator and rotor parts for inspection and maintenance without disassembling the stator. The 13·8 kV leads are brought out of the stator above the generator floor, rather than below the floor level, and run to the switchgear through a short metal duct which connects the generator housing directly with the switchgear cabinet. This arrangement provides the most direct and economical connection and also facilitates the stator end shift, as the main leads and control cables are easily accessible for disconnecting. The switchgear includes excitation equipment, 15 kV disconnect switches, potential and current transformers, surge protection, and grounding transformers. The generators are provided with automatic carbon-dioxide fire extinguishing systems.

The main cables from the generator switchgear to the transformers are supported on racks within the power station, and are brought up through a vertical shaft, about 50 m. long, in which they are suspended from the top by specially constructed bore-hole terminals, and braced but not supported along the shaft wall. This shaft is adjacent to the elevator shaft, and from it the cables are carried to the switchyard in a tunnel.

Transformers

Each generator is connected to its own 13·8/115 kV three-phase delta-star transformer located in the outdoor switchyard just downstream from the intake and directly above the powerhouse. The transformers have a self-cooled rating of 15,000 kVA and a forced air-cooled rating of 16,667 kVA with fans which are energised automatically upon rise in oil temperature. If temperature continues to rise, a relay operates an alarm, and on further rise opens the circuit breaker. Other protective fittings include a Buchholz relay, a tap changer interlock switch, and an oil thermometer and oil-level gauge equipped with alarm contacts. The transformers weigh about 35 tons each without oil, and were the heaviest single items shipped to Guayabo.

Station Service Unit

Service power is provided by a small hydro-electric generating unit, provision being made for the main generators to supply the service power, through a 500 kVA transformer, when the auxiliary unit is not operating. It consists of a horizontal-shaft Francis turbine rated 735 metric horsepower at 720 r.p.m. under a net head of 50 m., driving a generator rated 625 kVA at 0·80 power factor, 460 volts, three-phase, 60 cycles. It has two pedestal bearings, one of which is a combined journal and thrust bearing. The exciter is coupled directly to the generator shaft. The governor has an electric governor head similar to those in the main turbine governors.

Station Auxiliaries

The mechanical and electrical auxiliaries which provide the necessary station services and aid in the proper functioning of the main generating units are generally similar to those usually found in hydro-

electric stations. A raw water piping system drawing its supply from the spiral cases of the main turbines and from the penstock of the station service unit provides strained water for cooling the machinery. Treated water for drinking and general power station use is supplied through a separate piping system from a treatment plant in the switchyard area. Compressed air for general station use is supplied by a 25 h.p. water-cooled air compressor. The governor oil pressure tanks are charged with air from small compressors mounted at the governors. Fire protection for the powerhouse is provided by portable carbon-dioxide fire extinguishers, and for the main transformers and switchyard manually operated dry chemical extinguishers are available.

Station drainage is of special importance in an underground station, and requires liberal design in view of the uncertainties of estimating the amount of seepage that will enter, especially through the various auxiliary excavations such as the access tunnel, the elevator and cable shafts, etc. Guayabo is provided with a large sump which collects seepage water and equipment drainage and is also used for draining the turbine spiral cases and draft tubes during their unwatering. Three automatic pumps are provided for raising the sump water to the tailrace tunnel, with space provided for adding a fourth unit if necessary. Disposal of sewage from the station sanitary facilities is effected by a pump which discharges the effluent from the septic tank into the tailwater tunnel. The problem of venting the sanitary fixtures was solved by running a vent pipe to the cable shaft where there is a large quantity of ventilating air discharging to atmosphere.

The tropical climate at Guayabo involves problems of high temperature and humidity. The station is ventilated by air which enters through the main access tunnel, passes through the station and up through the cable shaft, at the top of which it is exhausted by two axial-flow fans rated 640 cu. m. per min. each. Normally one fan is in operation, the other starting automatically under thermostat control when the temperature at the top of the cable shaft reaches 40° C. corresponding to the cable temperature rating. It has not been found necessary to dehumidify the incoming air at Guayabo, as there is no objectionable condensation of moisture along the rock walls of the power station, which are relatively warm at this geographical location. However, all electrical control switchgear cabinets have small electric heaters to prevent moisture condensation inside. The control room is cooled for operator comfort by an air conditioning unit of 5-ton cooling capacity.

A combination passenger and freight elevator provides convenient access to the power station from the switchyard. This elevator can also be used for inspecting the adjoining cable shaft through removable side panels.

A 60-ton bridge crane, supported on concrete girders and columns placed along the rock walls of the generator hall, handles the heavy equipment parts. To reduce headroom requirements and minimise power station excavation the crane has two trolleys and the bridge girders are spaced to permit a generator rotor to be lifted into the space between them. A 10-ton monorail hoist in the tailrace surge chamber handles the 5-m. square vertical slide gate provided for the draft tubes.

The electrical auxiliaries at Guayabo generally follow conventional practice. Direct current at 120 V for control circuits and emergency station power and lighting is supplied by lead-acid storage batteries of 400 Ah capacity, which are charged by two 7·5 kW motor-generator sets operated on alternate floating schedule. A low-resistance earthing system is obtained by use of ground rods and plates in the reservoir bed and ground rods in the switchyard, all connected to a test station and to a copper grid in the power station to provide an earth for all non-current-carrying metallic parts in the station.

Station lighting is provided by a 220/110 V 3-wire single-phase system. A combination of high bay and ceiling lighting is used with good results in the generator hall. The control room has fluorescent lights and emergency ceiling floodlights. Power for the project village is supplied through a 300 kVA transformer in the switchyard.

Telephone communication between the power station control room, switchyard, substations, and the CEL office in San Salvador is provided by an automatic dial-type telephone system operating on a power line carrier current circuit of the single-frequency type, with amplitude modulation. A separate intercommunication telephone system serves the power station operating and maintenance personnel.

The power station is operated from the main control room located above the generator hall floor at the far end of the station, as seen in Fig. 1. A large window affords a good view of the generator hall from the control room.

Performance Tests

Official acceptance tests of the main turbines and generators were performed in December of 1954, on No. 1 unit. The turbine tests were conducted in accordance with the rules of the Power Test Code of the Amercian Society of Mechanical Engineers, using the Gibson pressure-time method of flow measurement. This method was selected in view of its manifest suitability for turbines with penstocks and because it combines the essential qualities of accuracy, convenience, and economy. Piezometers for applying the Gibson method with differential pressure diagrams were provided in the vertical penstocks at two gauging sections 16·46 m. apart, each section having four piezometer taps.

Power output was determined by electrical measurements at the generator terminals, and the generator losses were added to obtain turbine output. Net head during the test varied between 54·85 and 56·13 m. The results were corrected to a 56 m. head

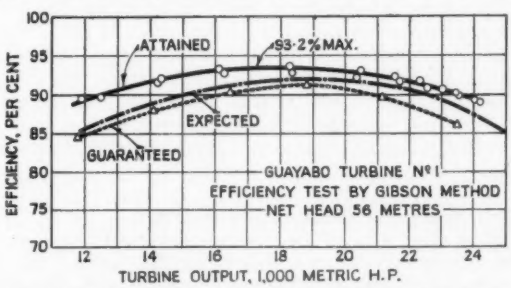

Fig. 10. Comparison of guaranteed, expected and attained efficiencies of Guayabo turbine

for which turbine contract guarantees had been made.

The test results (Fig. 10) show that the turbine power and efficiency exceed the guarantees by ample margins. Generator efficiency also exceeds the guarantees at all loads. The maximum turbine efficiency of 93·2 per cent. is considered very satisfactory for a turbine of only medium size. The flat shape of the efficiency curve and the high part-load efficiencies are very desirable characteristics when the units operate with relatively low load factors. The high efficiency attained can be credited to good design and workmanship by the turbine manufacturer, and to the favourable turbine setting selected for the Guayabo installation. It is seen from Fig. 10 that the attained efficiency not only exceeded the guaranteed efficiency, but also substantially surpassed the expected efficiency predicted by the manufacturer on the basis of a normal installation. It is believed that the horizontal setting, which assured excellent hydraulic conditions and enabled the economical use of long straight conical draft tubes instead of the usual elbow type draft tubes, resulted in a gain in turbine efficiency of the order of one per cent.

The total head loss in the pressure conduit including intake, transition, bend, and vertical penstock, was determined during the Gibson test to be 0·36 m. with turbine discharge corresponding to the rated output of 21,000 h.p.

Surge Chamber and Tailrace Tunnel

A horizontal tailrace tunnel about 270 m. long carries the turbine discharge back to the river channel downstream of the rapids. A single tailrace tunnel serves the first three turbines, but provisions have been made for a second tunnel to be driven when the two future units are added. A surge chamber has been excavated above the draft tube outlets to reduce pressure surges in the tailrace tunnel and to improve the regulation of the turbines. It is of the simple type and is a partially operating surge chamber designed to function only when the tailrace tunnel is under pressure due to high tail-water level caused by a flood. The chamber is vented to atmosphere through a combined vent and access shaft.

The tailrace tunnel is unlined and sized to operate normally with a free surface and a flow velocity of about 2·2 m. per sec. when all the turbines are carrying normal load. This results in a tunnel 7·8 m. wide by 9·36 m. high. A short discharge channel with concrete training walls, bottom pavement, and slots for stop logs is provided at the outlet portal.

Access Tunnel

The surface topography favoured an inclined access tunnel with an entrance above any possible rise in the tailwater level; this has been driven at a moderate grade (12·2 per cent.) to accommodate truck transport. The use of an access tunnel, rather than a vertical access shaft, greatly facilitated transportation of excavated material, construction, and subsequent operation of the station.

Switchyard and Transmission System

The main 115 kV switchyard contains three power transformers, one of which originally served as a spare for the two initial generating units, subsequently becoming the third unit transformer. Space is provided for additional transformers and switchyard equipment for the future units when these are installed.

The high-voltage terminals of the transformers are connected through oil circuit breakers and disconnecting switches to a 115 kV main bus which is connected through an air-break-line disconnect and earthing switch to the transmission system shown in Fig. 2. A single-circuit 115 kV out-going transmission line 61·6 km. long connects the Guayabo switchyard through a substation in San Rafael Cedros with the substation in San Salvador, where voltage is stepped down to 22 kV for subtransmission and distribution to the city. The San Rafael Cedros substation is connected by a 69 kV branch line 82·6 km. long with a receiving substation in San Miguel. All transmission towers are of steel, and the conductors, of aluminium with steel reinforcing, are arranged in triangular configuration. The ruling span on the 115 kV line is 457·2 m., the maximum being 9.45 m. For the 69 kV line the ruling span is 198·1 m., the maximum being 613·7 m.

Project Personnel and Principal Contractors

The Guayabo project was originally conceived and promoted by the late Eng. Rafael E. Lima of San Salvador. The project is owned and operated by the Comision Ejecutiva Hidroelectrica del Rio Lempa of which Victor de Sola is president. Complete engineering services were supplied by the Harza Engineering Company of Chicago. The Resident Engineer for Harza was C. E. Wattles, Eng. Enrique R. Lima, who is currently the Executive Director, acting for the CEL. The principal contractors were:—

General construction	— J. A. Jones Construction Co., U.S.A.
Transmission lines	—Techint, Italy
Turbines and governors	—Theodor Bell & Co., Switzerland
Generators	—Brown Boveri & Co., Switzerland
Power station crane	— Theodor Bell & Co., Switzerland
Gates and hoists for spillway and intake	— Stahlbau Rheinhausen, Germany
Switchgear	— Westinghouse Electric International Co., U.S.A.
Circuit breakers	— Ateliers de Construction Oerlikon., Switzerland
Transformers	— Ateliers de Constructions Electriques de Charleroi, Belgium

Costs and Financing

The cost of the project to date, with three 15 MW sets installed, has been $22·3 million. Of this $12,545,000 was obtained by a loan from the World Bank. The loan term is 25 years and interest rate is 4¼%. The remainder was raised by a revenue bond issue in El Salvador, together with certain grants from the Government of El Salvador. The local bond issue was the first of its type ever offered in El Salvador and the bonds were purchased quite rapidly by a large number of individuals in that country.

Prestressed Concrete. McCalls Macalloy Limited, Templeborough, Sheffield, have published a valuable booklet entitled the "Macalloy Manual," which is really a vade-mecum to the design of prestressed concrete structures. It contains separate chapters on design, detailing, stressing and applications. A section is devoted to tables giving data on properties of beams, prestressing forces, anchorage components, and other relevant matters, and appendices discuss high-strength concrete and physical properties of and specifications for Macalloy bars. The booklet contains 80 pages and is a quality production.

A Plan for the Yukon

We publish an abstract of a paper presented to The Institution of Civil Engineers by J. M. Wardle, C.B.E., B.Sc., M.E.I.C., describing a plan for a major power development in the Canadian Northwest

IT has long been known that Ventures-Frobisher-Quebec Metallurgical Industries Limited have been investigating the possibilities of a major power development in the Canadian Northwest, and an outline of the proposals has now been given in an important paper* recently presented to the Institution of Civil Engineers by Mr. J. M. Wardle. After considering various alternatives, the Yukon was selected as the most suitable river for development for the purposes envisaged, and the scheme proposed has an ultimate potential of nearly five million horsepower, which can be developed in four stages. The scheme complies with the requirement of the Canadian Government that the civil-engineering works, power stations, and the industrial areas to be developed shall all be within Canadian territory.

The Yukon River is an international stream, with its source in Canada and its mouth in Alaska on the Bering Sea. Its total length, including the large lakes at its source, is 1,700 miles, of which 570 miles are in Canada. The Canadian section is under two jurisdictions since the 60th parallel of latitude, which constitutes the boundary between the province of British Columbia and the Yukon Territory, crosses the upper lakes. Some 4,300 sq. miles of its Canadian watershed is in British Columbia under provincial administration and the remainder is in the Yukon Territory, which is under the control of the Government of Canada. The Territory is situated in the northwest corner of the Dominion and to the north it extends nearly to the 70th parallel and is bounded on the north by the Beaufort Sea. It has an area of 207,076 sq. miles, of which 109,500 are in the Yukon River watershed (Fig. 1).

For many years the Yukon River was not regarded as a major power source but as the need for low-cost hydro-electric power steadily increased, consideration was given to what the Yukon River watershed might have to offer. Interest in this area was heightened by the existence of several large fresh-water lakes at the head-waters of the river, with a total surface area of over 640 sq. miles. The possibilities of securing the essential storage required for a northern power development were thus very great and constituted one of the main assets of the Yukon River basin.

Two courses were open for the development of power in Canada from Yukon waters. The first of these, a series of large dams down the Yukon River, was not regarded as economically feasible. The alternative, which complied with Canadian Government policy requiring development in Canada, was to reverse the northerly flow of the upper waters and take these south through the lower end of Atlin Lake to low valleys in British Columbia where heads from 500 ft. to over 1,000 ft. were available. This is the general plan that was adopted and for which field surveys and engineering investigations have been under way since the early summer of 1953.

The power plan adapts itself readily to stage construction, one of its most important assets. Power output can be planned to keep in step with anticipated markets, and beginning with 250,000 h.p. can ultimately, through four major stages, reach a maximum of 4,900,000 h.p. These stages are outlined below.

Stage One

The large upper lakes which include Atlin, Lindeman, Bennett, Tagish, Taku Arm, and Marsh Lakes have elevations ranging from 2,192 ft. to 2,147 ft. above sea level and it is proposed to raise all these lakes to a F.S.L. (full surface level) of 2,215 ft. by a dam a few miles above Whitehorse. The large reservoir so created will extend southerly in three great arms ranging in length from 50 to 120 miles.

The mean flow of the Yukon River at Whitehorse over a 42-year period ranges from 8,600 to 8,800 cusecs, which is equivalent to 6,370,000 acre-ft. The reservoir capacity between a proposed L.S.L. (low surface level) of 2,195 ft. and F.S.L. of 2,215 ft. is estimated at 6,814,000 acre-ft.

This water will flow through Atlin Lake to a tunnel which will carry it to Sloko Lake. The latter will be used as a power canal to carry the water to the portal of tunnel No. 2 as shown in Fig. 2. This tunnel, which will be 10 miles long, will discharge the water into a power house on the Nakonake River under a gross head of 1,090 ft. An estimated 880,000 h.p. will be developed here. An unusual feature is the use of Sloko Lake as a canal since its elevation is 190 ft. higher than Atlin Lake. Consequently, unless when lowered its carrying capacity would be 28,000 cusecs or more, the redesigning of the general tunnel system would be necessary.

To obtain this essential information a survey party took careful soundings through the ice cover in March 1954, and these, when plotted, showed that the minimum cross-section of Sloko Lake when drawn down from 2,382 ft. to 2,192 ft., could carry 28,000 cusecs at a velocity of 1 ft./sec.

Stage Two

This consists essentially of increasing the amount of water available in the reservoir by the addition of water from Teslin Lake and River which lie to the north-east of Atlin Lake. Teslin is a very large lake and the mean flow of the river which drains it is 11,800 cusecs. Its flow is thus considerably larger than the

* "A Major Power Plan for Yukon River Waters in the Canadian Northwest." I.C.E. Paper No. 6213. The paper is also to be read at the Annual Meeting of the Engineering Institute of Canada, in Banff, in June 1957. Mr. Wardle was for 14 years Director of Engineering and Water Resources for the Dominion of Canada, and is now Consulting Engineer for Ventures-Frobisher-Quebec Metallurgical Industries Limited, Ottawa.

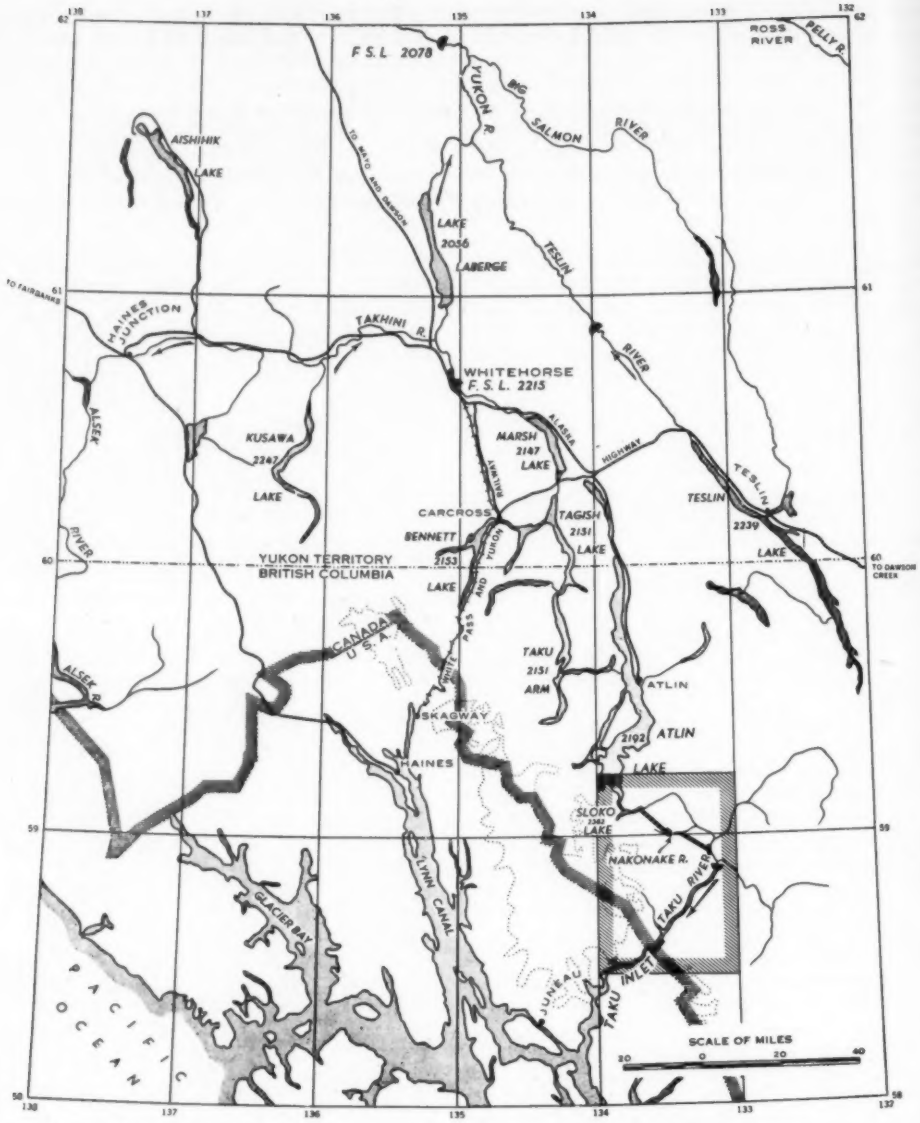

Fig. 1. Map of the Yukon catchment, showing the development area

flow of the Yukon River at Whitehorse. It is proposed to regulate this flow by a storage dam at the mouth of the Mary River some 35 miles below the outlet of the lake so that a minimum of 6,000 cusecs can be diverted into the upper reservoir. This diversion will involve power canals and a tunnel.

In 1955 a survey of the diversion was made, but the economic study has not yet been completed. This may show that a low-lift pumping plant at the summit of the diversion may be preferable to longer canal and tunnel sections. The net benefit from the diversion can be improved by increasing the amount of water diverted from the Teslin River watershed. With a minimum of 6,000 cusecs diverted into the upper reservoir at Marsh Lake, the power output at the Nakonake River power plant will be increased to 1,540,000 h.p.

Stage Three

An advanced stage of development will be reached at this point when it will be possible either to utilise a head of 550 ft. that is available between the Nakonake and Taku Rivers by means of an eight-mile tunnel, or to impound waters below, i.e., north of the Whitehorse dam and thus create a lower reservoir.

On the assumption that it would be more economical to increase to the maximum the water available at the high head of 1,090 ft., the creation of the lower reservoir seems preferable. Another important advantage is that the flooding of the town of Whitehorse will be avoided. These factors far outweigh the loss of power required to pump water from the lower to the upper reservoir with a maximum lift of 140 ft.

The lower storage dam on the Yukon River will be built below the mouth of the Big Salmon River and

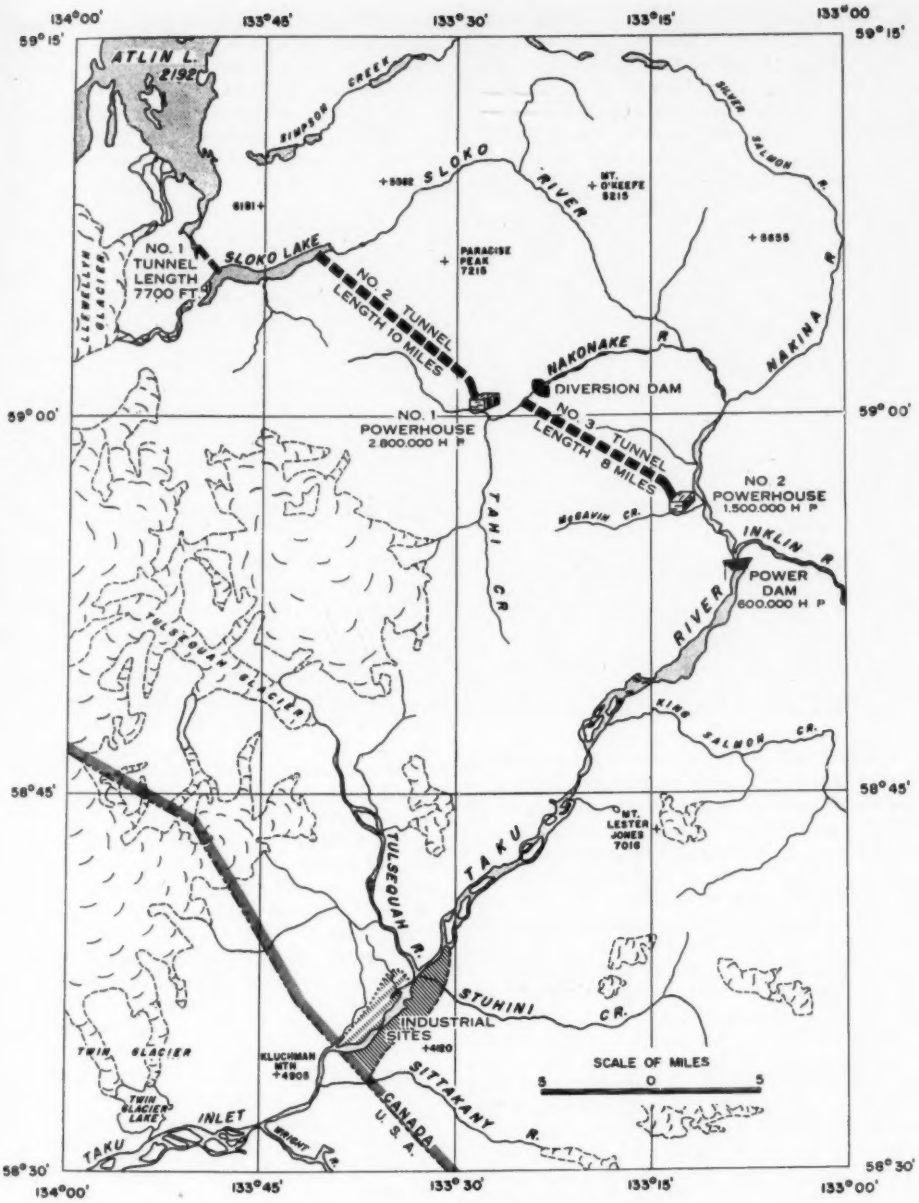

Fig. 2. Map of the proposed development

will raise the level of the river channels, Lake Laberge, and smaller lakes, to an elevation of 2,078 ft., which is the normal high water mark at Whitehorse. This reservoir would impound waters from Yukon tributaries below Whitehorse, including the Teslin River waters not already diverted, and the waters of the Takhini and Mendenhall Rivers.

Upper waters of the Alsek River to the west would be reversed by a dam on the Dezadeash River to flow into the Mendenhall River through a diversion surveyed in 1953. These waters, together with those of the Big Salmon, would give a total regulated flow at the south end of Atlin Lake of 28,000 cusecs. The total output at the Nakonake River power plant would, under Stage Three, be increased to 2,800,000 h.p. If necessary at some time in the future, water from the Kluane Lake watershed, about 125 miles west of Whitehorse, and which now flows northerly, might be diverted to the lower reservoir. While at least 2,000 cusecs are available here, the diversion would not be economical in the earlier stages of the proposed development. However, its feasibility would gradually be improved as increased power output permits a greater distribution of costs.

Stage Four

Under present plans the next step would be the driving of the eight-mile tunnel between the Nakonake

and Taku Rivers, as shown in Fig. 2, which would create a head of 550 ft. In the meantime the estimated flow of 28,000 cusecs at the south end of Atlin Lake, as available from Stage Three, would be increased by the flow of the Sloko Lake watershed and upper Nakonake River, besides one or two smaller streams of a tributary nature. An estimated 1,500,000 h.p. can thus be developed at the plant on the northwest bank of the Taku River.

During 1956 a promising dam site was located on the main stream of the Taku River just below the mouth of the Inklin River. Subject to detailed surveys the flow of the Taku and Inklin Rivers could be regulated here to give, with the northerly waters, a flow of 32,000 cusecs. A power plant at the dam site would give approximately another 600,000 h.p., or a total potential of 4,900,000 h.p.

Tunnels

As shown in Fig. 2, three tunnels will be required to bring water to the generating plants, the first of these, No. 1, carrying water on an even grade from the lower end of Atlin Lake at Sloko Inlet to Sloko Lake. No. 1 tunnel will be 7,700 ft. long. No. 2 tunnel will carry water under head from the east end of Sloko Lake to the generating plant on the Nakonake River, and will be from 10 to 10·2 miles long. No. 3 tunnel, which will carry water from a diversion dam below the Nakonake power plant to the proposed generating plant on the Taku River, will be approximately eight miles long.

Several factors enter into tunnel planning and design. If Yukon River power was within transmitting distance of a major grid with an assured demand, it would not be difficult to decide on the capacity of the initial structures. However, power loads in the area are negligible, and a market must be developed based on mining, and the processing of raw materials in large quantities. To carry a flow of 28,000 cusecs at normal velocity would require an unlined tunnel with a diameter of about 60 ft. A lined tunnel with higher velocity would have a diameter of approximately 50 ft. The greater size of these tunnels and their length and cost prohibit their construction in the early stages of the development because no private company could arrange the financing with no earnings on the total investment for several years. Consequently smaller, pioneer, tunnels will first be built. As the power load develops, larger tunnels will be driven parallel to the pioneer tunnels, and with the same gradients, unless rock encountered by the latter shows the necessity of some change in alignment. The proposed diameter of the pioneer tunnels is approximately 31 ft.

Study has yet to be made in regard to the choice of lined or unlined tunnels, where the latter type are possible. From a construction standpoint only it might be economical to drive the larger unlined tunnel section, but there are other factors to be considered which can only be assessed when construction has commenced.

In No. 1 tunnel the rock types to be encountered will be basalt lavas, limestone and sandstone. When excavation is well advanced at the portals, the denser undisturbed rock will probably stand up, but its schistose composition and water-soluble content may prove to be a weakness. Lining will consequently be necessary through this type of rock. The other rock formations mentioned, though dense and compact, will be affected by continuous passage of water; solution cavities are bound to form. As these could lead to rock falls and cave-ins, it is expected that all of No. 1 tunnel will be lined.

In the case of No. 2 tunnel, at least two-thirds of its length from Sloko Lake southward will be in Cretaceous volcanic rock. The lower end in the Nakonake River valley will be in Jurassic sedimentary rock. These rocks are strong and no tunnel lining should be required. However, it is planned to line the lower end of No. 2 tunnel to prevent erosion in the limestone beds. It is not expected that any serious flows or ground-water pressures will be encountered; the centre section of the tunnel under the mountain ridge should be relatively dry.

Drilling has been carried on throughout the 1956 season at the inlet of No. 1 tunnel, both portals of No. 2 tunnel and at the proposed power-house site on the Nakonake River. The results do not show any great differences in expected rock formations, except at the upper portal of No. 1 tunnel, where a fairly long open cut may be necessary.

Industrial Sites

The selection of a suitable industrial site where large plants could be established to use Yukon River power required a good deal of study, but after considerable investigation a satisfactory industrial area was found on the south bank of the Taku River just east of the international boundary where about 1,100 acres of level ground are available with reasonable development costs. On the opposite side of the river there are another 4,000 acres which could be developed later should the cost be justified by industrial development.

Northwest Power Industries Limited, the subsidiary formed to investigate and carry out the development, hopes to market much of its power in the base-metal fields and in the fields of forest products, chemicals and plastics.

Probably no major development in the Canadian Northwest is linked more closely with, or will stimulate more, the development of Canada's natural resources. It will develop transportation facilities, encourage exploration and surveys of all kinds, and bring settlement and stability to unpopulated areas. Yukon power can be a prime factor in establishing an industrial empire in Canada's northern regions.

NOTE. Figs. 1 and 2 are reproduced from Mr. Wardle's paper by courtesy of the Institution of Civil Engineers.

Hydraulic-Engineering Structures. Fried. Krupp, Maschinen- und Stahlbau Rheinhausen, have published in English a 48 page brochure devoted to illustrations of hydraulic-engineering structures they have built all over the world. The number and size of the structures illustrated are truly impressive, and we are left with the impression that there is nothing in this field that this firm could not tackle. Weirs of all kinds are depicted, equipped with gates of the rolling, roller, sluice, stepped-edge, radial, sector, fishbelly-flap, and drum types. Syphon spillways and circular intake towers are also included. A large section of the brochure is devoted to gates and equipment for locks, docks, ship lifts, pontoons and floating docks.

Fig. 3. Aerial view of the site of the Serre Ponçon dam and the Serre de Monge gorge

The Durance Project

This article is devoted to a description of the Serre Ponçon dam
by Peter O. Wolf, B.Sc.(Eng.), A.M.I.C.E., A.M.I.W.E.

PART TWO

As stated in Part One,* the reservoir at Serre Ponçon is situated at the downstream end of the Upper Durance and extends up the Durance and the Ubaye valleys. Its catchment area of 3,600 sq. km. has a mean altitude of 2,020 m. and includes the areas receiving the highest annual precipitation as well as the main glaciers feeding the Durance. The original river level at the dam site was approximately 670 m., and the top water level of the reservoir will be at elevation 780 m.

The Upper Durance is about 75 km. long and, with a slope of 1 in 70, has the characteristics of an upland river with widely varying discharges. The mean annual flow in the Durance downstream of the mouth of the Ubaye is 2,670 million cu. m., equivalent to a mean discharge of 85 cu. m. per sec. or 3,000 cusecs. Approximately 4½ km. downstream of the Serre Ponçon dam, the River Blanche joins the Durance from the south. An intake on the Blanche, at elevation 800 m., will divert an annual flow of 100 million cu. m. through a tunnel of nearly 5 km. into the new reservoir.

As explained in Part One, the need for the Serre Ponçon reservoir arises primarily out of the wide variation in discharge, from drought to flood, and the storage of 1,200 million cu. m. (of which 900 million or one-third of the mean annual flow, will be live storage) will go far to ensure the maintenance of the even flow so urgently needed for reliable water supplies, whether for irrigation or power production. The reservoir will be able to absorb most floods from the Upper Durance, so giving much needed relief along the middle reach and—more important still—in the fertile areas along the Lower Durance.

The need for such a huge storage volume compelled the designers to choose a structure of greater height than they would have desired. Very careful studies of the economics and of the social effects of the scheme preceded the decision to adopt such a vast scheme which, by creating an artificial lake of more than twice the size of the Bort reservoir (previously the largest in France), will drown the two

* WATER POWER, Vol. 9, No. 4, April 1957, p. 146.

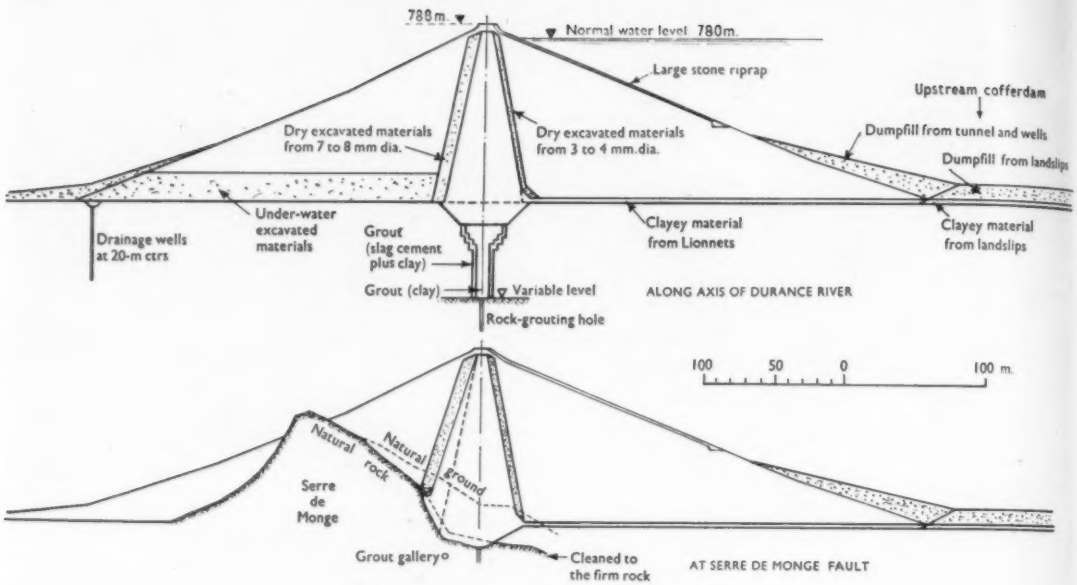

Fig. 4. Section through Serre Ponçon dam

villages of Savines and Ubaye, 28 sq. km. of land, 15 km. of railway and 25 km. of road.

The bottom 300 million cu. m. of the reservoir will be available for the sediment deposition which will inevitably occur in upland rivers like the Upper Durance and the Ubaye. Of the live storage, 200 million cu. m. initially, and 300 millions ultimately, will have to be made available for the water supply (chiefly for irrigation) of the areas downstream.

In 1912, Mons. Wilhelm put forward a project mainly for irrigation and flood protection, which is now referred to as " Petit Serre Ponçon." He proposed a top water level at elevation 752 m., a total storage of 600 million cu. m. of water and a reservoir area of 16·5 sq. km., which would have avoided the submergence of Savines.

" Petit Serre Ponçon " was recently recalculated. The live storage in the reservoir would have been largely reserved for water-supply purposes, with power production reduced to a small output of little reliability. On the other hand, the first cost of re-establishing roads and railways, and of the costlier part of the Serre Ponçon dam (the cut-off works in the alluvium, to be described later), would not have been much less than with the present scheme. The first cost of " Petit Serre Ponçon" was estimated to be 75 per cent.* of that of the present scheme, whereas the revenue from power production would have been reduced much more because of the relatively small annual output of unremunerative " spill " energy, produced at a lower head.

Incidentally, the reservoir area of Petit Serre Ponçon would have covered the most fertile agricultural land of the farmers of Savines, so compelling many of them to leave the area, and the local inhabitants have accepted the present scheme with good grace.

* A. Decelle, J. Cabanius, and M. Gueiton, "The Development of the River Durance," Supplement to *Travaux* No. 247, Paris, May 1955, on the occasion of the Fifth International Congress on Large Dams.

Preliminary Investigations

The concept of providing protection from floods on the Middle and Lower Durance by the construction of a large flood-storage reservoir at Serre Ponçon goes back more than a century. After the disastrous floods of 1843 and 1856, studies were undertaken which not only pointed to the present dam site as the most suitable one (a relatively narrow valley at the downstream end of a large basin) (see Fig. 3), but even included the first test boring. After the drought of 1895, site investigations were resumed under the inspiration of Mons. Ivan Wilhelm (Ingénieur en Chef des Ponts et Chaussées). In order to find the position of the bed-rock, a hole was sunk through the alluvium and this revealed a depth of soil of over 42 m. In 1912 a shaft was sunk through the rock on the right bank, with an adit driven from its foot towards the centre of the valley indicating a depth of alluvium of over 50 m. Large quantities of water at 140°F., with a high content of sulphurous and other minerals, entered the adit from the alluvium, and the first multiple-purpose scheme conceived by Mons. Wilhelm and his colleagues to provide hydro-electric power, irrigation supplies, and a flood-storage reserve, had to be abandoned. After the war of 1914–18 new investigations were undertaken which included deep borings. Most of these reached solid bed-rock, but some did not, so indicating that the depth of the alluvium in the buried cleft exceeded 90 m.

In 1947 Electricité de France took over the detailed survey of the site and carried out borings along several sections of the valley. Numerous holes were bored from the surface, some vertically and some obliquely, and a number were driven from shafts, at various angles and starting from various levels, so giving a well defined picture of the bed-rock over a length of over 1 km. along the valley. The buried valley was found to be approximately 110 m. deep and to divide

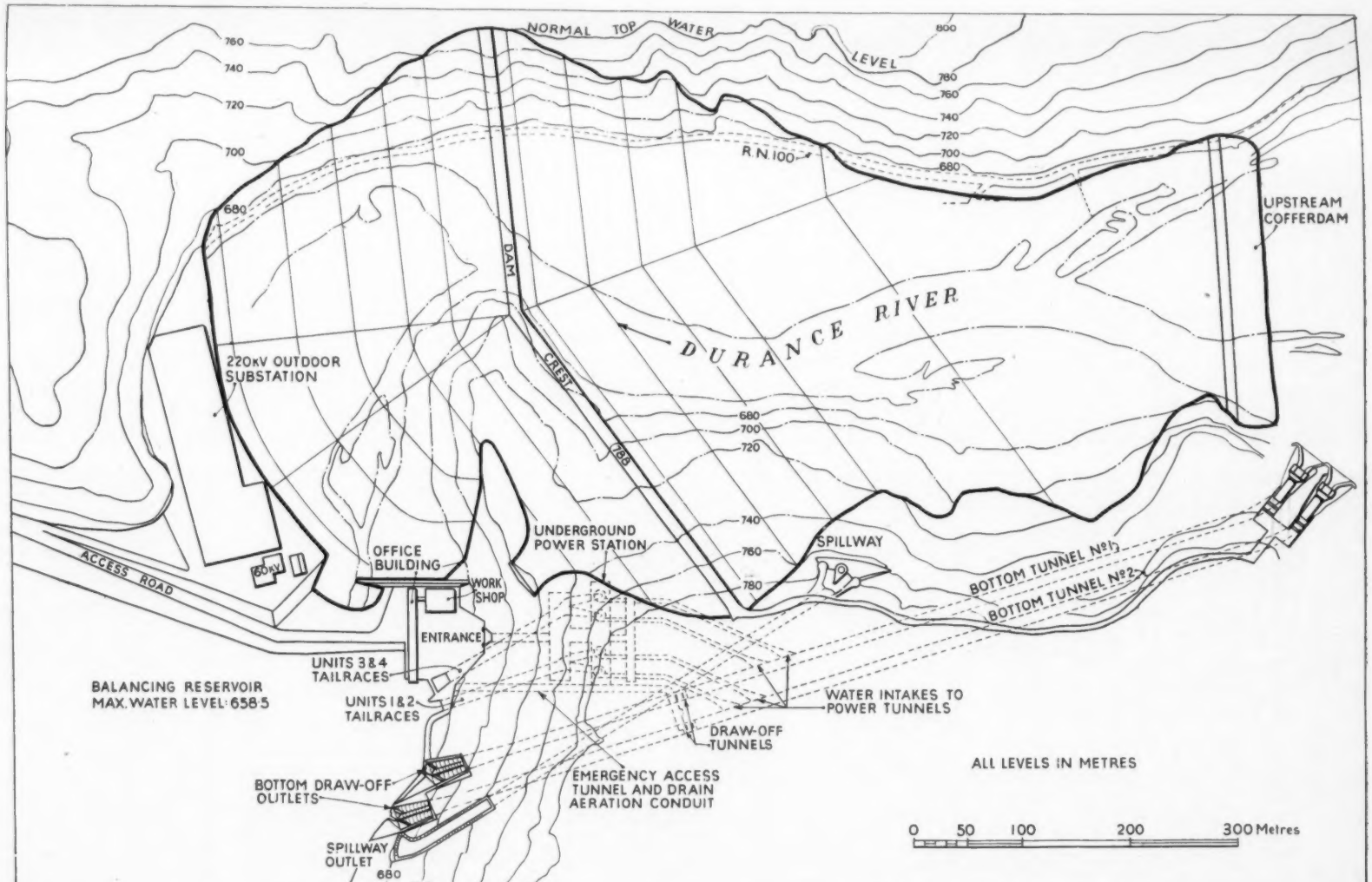

Fig. 5. Plan of Serre Ponçon dam and power station

into two clefts just upstream of the axis of the dam. Geophysical methods were used to check the picture obtained from the results of the borings.

Throughout the history of the investigations, the intention had been to construct a masonry dam, and even in 1947 the engineers of Electricité de France were thinking in terms of a mass-concrete structure.

The borings of 1947–48 again tapped large quantities of hot sulphurous water, under a head corresponding to the water level in the River Durance, and it was clear that a concrete dam would require special protection from chemical attack.

The 1947–48 investigations indicated that the alluvium consisted of irregular lenses of sand and gravel with size distributions varying from one point to the next, but all of high compressive strength and unlikely to give appreciable settlement under large vertical stress. The permeability also varied from point to point, the range being generally from 1 to 10×10^{-2} cm. per sec. Most tests showed the alluvium to be anisotropic, with the horizontal permeability about four times higher than the vertical permeability. The alluvium near the bed-rock was more permeable, however (up to 200×10^{-2} cm. per sec.), and the fissured surface layer of the rock permitted considerable percolation. The flow of ground water below the river was estimated at 200 litres per sec. (or 7 cusecs).

An estimate of the cost of excavation down to sound bed-rock, and of constructing a concrete dam from a foundation level of 100 m. below the old river bed, indicated that the design originally envisaged —a mass-concrete dam—was uneconomical. Although large earth dams are unusual in France, the idea of constructing a dam of compacted alluvium was, therefore, investigated in 1948. In view of the great strength of the alluvium and the availability of material of low permeability in the vicinity, the preliminary studies led to the conclusion that an earth dam would give a far more economical scheme. Intensive research was, therefore, carried out on the problems of earth-dam design and construction, during the period 1949–54, and extensive use was made of the experience gained in other countries, particularly the United States.

The design of the watertight membrane in the dam and through the alluvial foundation was carefully considered. Both reasons of cost and need for structural flexibility led the designers to abandon the idea of a reinforced-concrete slab. It was decided to use the local clayey materials for the core of the dam, and to waterproof the foundation by grouting up a vertical curtain.

At this point the possibility was considered of building a concrete dam on a part of the alluvium thoroughly consolidated by grouting, but the earth dam was again found to be preferable.

The Design and Construction of the Serre Ponçon Dam

The dam will rise 120 m. (400 ft.) above the river bed, so ranking with the highest earth dams constructed so far. Its width is 650 m. (2,170 ft.) at the base, and the crest will have a length of 600 m. (2,000 ft.). The total volume of fill will be over 14 million cu. m., of which 2 million will consist of argillaceous material in the watertight core and 12 million of Durance alluvium supporting the core on both sides; between the core and the supporting fill, finer grained alluvial materials will be interposed to act as filters retaining the material of the core when subject to seepage pressure gradients. The upper part of the upstream face will be protected by 100,000 cu. m. of large block pitching against damage by wave action (Fig. 4), the thickness of the pitching being 1 m. and the average size of block 0·1 cu. m.

The watertight core increases in thickness from a few metres just below the crest level of 788 m. to 55 m. at the bed level of the Durance. Below that it narrows down to 35 m. at the bottom of the grouting trench from which clay and cement grout is injected into the alluvium through several rows of grouting holes to form a grout curtain 15 m. (50 ft.) thick. Grouting galleries are also to be driven through the bed-rock, in line with the watertight core and approximately 20 m. from the rock surface, and grout injected from these galleries is intended to tighten up the top 20 m. of rock.

The supporting fill of Durance alluvium is obtained from a borrow pit downstream of the dam, where there will be a balancing reservoir. The proportion of grains smaller than 5 mm. is only 20 to 40 per cent., and compacting tests have proved the soundness of the construction technique: the fill is placed in layers of 50 to 60 cm. thick, and 10 per cent. by volume of water is then sprayed over it. Two passages backwards and forwards of a 45-ton pneumatic roller (tyre pressure 90 lb. per sq. in.) suffice to obtain a maximum dry density of 2,250 kg. per cu. m. (140 lb. per cu. ft.). In 1953, the U.S. Army Corps of Engineers carried out tests on samples from the Durance borrow pit and found the angle of internal friction to be 41°.

The argillaceous material for the core is obtained from the deposits at the junction of the Lionnets Torrent which discharges into the Durance, from the north, a few hundred metres upstream of Serre Ponçon. There is considerable variability of the grain-size distribution, but after suitable wetting, eight passages backwards and forwards of a 19-ton sheepsfoot roller over layers of 20 cm. have been found to give satisfactory compaction to 116–125 lb. per cu. ft. The resultant permeability is of the order of 10^{-6} to 10^{-7} cm. per sec. The angle of internal friction has been found to exceed 30°, but in stability calculations a value of 28° has been adopted.

The grout used for forming the cut-off in the alluvium has to seal both sands and gravels. A mixture of colloidal clay (for fine materials) and slag cement with 2 to 3 per cent. of caustic soda acting as a setting agent (for quick-setting action in open layers) has, therefore, been adopted, and results in a reduction of permeability from 10^{-2} to 10^{-5}, and even 10^{-8} cm. per sec.

The power station, together with the intakes, power and tail-race tunnels, gated bottom draw-off, and high-level flood spillway, will all be underground, hewn out of the excellent rock of the Serre de Monge (Fig. 5).

The tail-race tunnels will discharge into a balancing reservoir whose top water level will be maintained between 651·5 and 657·5 m. by means of a gated barrage near Pont d'Espinasses, and whose storage volume will amount to 6 million cu. m.

The maximum net head on the machines will, therefore, be approximately 125 m., with a mean of 107 m. The power tunnels, which are fed from the original temporary diversion tunnels, are designed for

a flow of 300 cu. m. per sec. There will be four vertical Francis turbines, of 80 MW each, which are expected to yield a mean annual output of 700 million kWh. The transformer station will raise the voltage to 150 or 220 kV.

The flow-regulating effect of the Serre Ponçon reservoir will enable the existing power stations on the Middle Durance to increase their annual output by a total of 250 million kWh, and to this will have to be added a further extra of 150 million kWh per annum to be produced by the future power stations for the same reason.

The flood-discharge arrangements for this earth dam have received especial attention. The estimate of the peak flood of 5,000 cu. m. per sec. will be discussed separately, but it may be said here that although the highest recorded flood peak reached only 1,800, such a large margin was considered prudent. The reservoir lag would reduce the peak inflow of 5,000 to a peak outflow of less than 3,200 cu. m. per sec. The main spillway consists of two gated inlets to a single circular tunnel provided with a deflector bucket at the downstream end. With the reservoir water level at 780 m. the discharge capacity of the arrangement will be 1,800 cu. m. per sec., and at elevation 782·5 m., it should be 2,000 (or 70,000 cusecs). The temporary diversion tunnels will be closed by valves which can be opened to permit a flood flow of 600 cu. m. per sec. each, through the draw-off tunnels.

Following the declaration of January 5, 1955, by the French Parliament that the Durance development was a matter of urgent national importance, Electricité de France arranged for four French firms of civil-engineering contractors to form a consortium, in association with an American company having experience constructing large earth dams, to do the work. Operations on the Serre Ponçon dam were started three months later, and are to be completed within the contract period of 39 months.

The area is in one of the wetter Alpine districts, and there will inevitably be much loss of placing time due to rain, snow and frost. The contractors' estimate, for that reason, was based on 225 working days per year.

The controlling item in the contract is the placing of the 12 million cu. m. of alluvium, as the rate of progress of the other items will have to be adjusted to keep in step with this main task. The first problem was to provide access, and the maintenance of good roadways to assist the vehicles to place the fill will be a major task.

At the same time work had to begin on the two concrete-lined diversion tunnels, each of 9·3 m. finished diameter, one 865 and the other 910 m. long, and each capable of discharging 750 cu. m. per sec. without drowning. As the downstream borrow pit is deepened, the water level in the Durance will fall and the discharge through the diversion tunnels will gradually increase, with increasing gradient, over the period of construction.

On completion of the diversion tunnels and of the upstream and downstream cofferdams, a very heavy placing programme will have to be faced. The contractors have planned their operations on the basis of there being thirty-four 300 h.p. Euclid bottom-emptying semi-trailers (of 25 cu. yd. capacity) on the site, and expect to move an average of 17,000 cu. m. per day with twenty-one of these machines available all the time.

Concrete is required mainly for the underground works. It will be batched at a central mixing plant, situated on the right bank, and sent to the site. The only noteworthy feature is that cement is handled in bulk containers discharged pneumatically.

NOTE.—The illustrations accompanying this article are reproduced from Mons. R. Giguet's paper "A French Dual-Purpose Scheme: the Durance Project and the Serre Ponçon Dam," presented to a joint meeting of the Institution of Civil Engineers and Ingenieurs Civils de France. *Proc. I.C.E.*, April 1957.

(To be continued)

ASCE National Convention

The American Society of Civil Engineers is holding its Annual Spring Convention in Buffalo, on Lake Erie, June 3-7, 1957. The focal point of the Convention will be the St. Lawrence Seaway and Power Projects. The technical programme for the four-day session features many papers and discussions concerning the design, construction and administration of the international engineering ventures.

The Power Division tours will include visits to the Sir Adam Beck hydro-electric station which includes two installations, the first completed in 1930 with a capacity of 525,000 h.p. and the second completed in 1956 with a capacity of 1,428,000 h.p., making 1,953,000 h.p. in all. An interesting feature of the latter plant is a pumped storage development, designed to accommodate substantial daytime peaking.

Technical sessions of the Power Division will begin on Monday afternoon, June 3rd, and be continued on Tuesday morning, June 4th; on June 5th papers will be given on penstock and anchor block design, fabrication and construction. Engineers presenting papers at these sessions are to be G. R. Latham, Ebasco Services, Inc.; I. E. Boberg, Chicago Bridge & Iron Company; J. Parmakian, U.S. Bureau of Reclamation; E. Schmitz, A. O. Schmitz Corporation; G. V. Richards, Pacific Gas and Electric Co.; and R. Bouchayer and S. Ferrand, both of Bouchayer and Viallet, Grenoble, France.

The Committee on Research, Hydraulic Division, is arranging a series of papers on model research conducted by Canadian and United States agencies for the St. Lawrence Seaway and Power Projects. W. L. Baines, National Research Council of Canada; E. B. Lipscomb, Waterways Experiment Station; Otto Holden and J. B. Bryce, both of the Hydro-electric Power Commission of Ontario, will present papers Thursday morning, June 6th. The afternoon programme of Thursday is sponsored by the Committee on Design. Papers presented during this session will treat design features associated with the regulation of Lake Ontario, the Grass River and Eisenhower Locks and the St. Lawrence Project power facilities. R. H. Clark, Department of Northern Affairs and National Resources, Canada; Franklin F. Snyder, Office of the Chief of Engineers; George E. Hyde, Uhl, Hall & Rich; Martin E. Nelson and Harvey J. Johnson, both of the St. Paul District, Corps of Engineers, will author papers for this session.

Cook Strait Cable Report

A technical report has been submitted to the New Zealand Government by British Insulated Callenders Cables Limited

DIRECT-CURRENT cables working at 250 kV have been recommended for the Cook Strait link between the New Zealand North and South Island power systems. British Insulated Callenders Cables Limited, who were engaged by the New Zealand Government in April, 1956, to investigate transmission of power across the strait, have reported on the project as "thoroughly practicable."

The initial proposal is to provide for the transfer of 600 MW—half as much again as the present South Island generating capacity—on a £16 million transmission scheme between Benmore in the South Island and Wellington. Of this £16 million the cable itself would cost nearly £3 million. The report takes into account the doubling of the transfer of power to 1,200 MW. The second stage of the cable link would cost about £2·1 million.

For the first stage it is recommended that the cable should run from Oteranga Bay, at the southern end of the North Island, to Fighting Bay, opposite it in the South Island, a distance of 25½ miles. Two cables and one spare would be needed for the first 600 MW direct-current link, compared with 12 cables and two spares for equivalent a.c. transmission.

The main points made in the report to the Government are:—

(1) There are several suitable routes for the cables. In the area recommended, the bottom of Cook Strait has been shown to be reasonably flat and largely free from rock.

(2) There are no precipitous cliffs or crevices and the maximum gradient does not exceed 1 in 5. There is a zone about seven miles wide, the depth of water in which is not greater than 900 ft.

(3) The rugged coasts limit the choice of landing sites for the cable. After the Oteranga Bay-Fighting Bay route, the consultants suggest Opau Bay in the North Island to Lucky Bay in the South Island for future cable-laying.

(4) The preferred area of cable laying is outside the Terawhiti and Karori tidal rips. One inactive earthquake fault line would be crossed but, in case of any movement, the conditions would be no worse than those the cables will have to stand in the normal course of laying.

(5) As the weather in the strait is very changeable the time for laying each cable must be less than 24 hours. It is estimated that the whole laying operation for one cable should take about 17 hours.

(6) Estimated time for the manufacture and laying of the three d.c. cables is 13 months, but much preparatory work will have to be done and proving tests made.

The cost of the complete transmission system from Benmore to Wellington would be about £16 million (direct current), compared with nearly £26 million (alternating current), said the Minister of Works, Mr. Goosman. The report, he said, had been given in full as a matter "of great public interest."

The report is founded on extensive site investigation by specialist engineers of the investigating company over a period of eight months, who thoroughly examined possible cable hazards. The report shows that they used underwater television as well as echo soundings to determine a path into Oteranga Bay. Television revealed a path up to 300 yards wide and with a smooth bottom down either side of the bay. They also undertook a comprehensive series of laboratory experiments to safeguard the possible laying of cable over rocks. A pinnacle of rock 20 ft. high with a small radius was taken as the extreme case.

Failure of the armoured gas-filled cables recommended by British Insulated Callenders Cables Limited for the Cook Strait crossing is "extremely unlikely," according to the report submitted to the Government. The cables, which are about 5 in. in diameter would be made in continuous lengths. The type had met all specified requirements and lent itself to ready handling and laying, said the report. There was no reason to suppose that the projected cables would not work with the same reliability which was expected for land cables. In the event of damage practical methods of repairs were available, and special repair techniques had been developed.

The cable should be laid by a fully equipped cable ship, the report recommended. Experienced engineers were available and there were suitable ships in Wellington with fully experienced crews which could be chartered for the laying operation. Throughout the routes it was expected that the cables could be laid on sediment. But to provide for the possibility of isolated rock out-crops the cable would be armoured.

Before the main manufacture begins the company unreservedly recommends the laying of a trial length of cable in a selected area to give about two years' experience of the worst conditions. The trial length itself could be made and laid within 18 months.

The only possible criticism of direct-current cables was their effect, when loaded, on ships' compasses, the report added. This aspect had been investigated and, on the contemplated routes, the effect, due to the orientation of the cables, was "negligible."

When the transfer of power of more than 1,200 MW was needed, a new route would have to be developed in the case of the alternating-current system. Such a route was the long one (41 miles) between Lyall Bay and White Bluffs.

On the other hand, with the few cables involved in the direct-current system, there would be ample room over either of the selected routes, if they were developed as suggested, to install cables for any foreseeable extra power.

The following table of comparative costs for the two cable systems is included in the report to the Government by British Insulated Callenders Cables Limited. Each is an estimated figure of the installed cost in pounds sterling, at December 1, 1956:—

	D.C.	A.C.
1st stage 600 MW	£2,984,000	£11,964,000
2nd stage 600 MW	£2,179,000	£12,300,000
Total 1,200 MW	£5,163,000	£24,264,000

An Unusual Power Plant

A number of novel features have been introduced into a small hydro generating unit situated in Mullardoch Tunnel, which forms the connecting passage between two storage basins

By P. J. E. STRIDE, B.Sc.(Eng.), A.M.I.Mech.E.

THE Mullardoch tunnel hydro-electric plant forms part of the North of Scotland Hydro Electric Board's Glen Affric Scheme. It was commissioned in December 1955 and is of considerable interest as it is the first large induction generator unit to go into operation in Great Britain. It is also the first fully underground hydro-electric station reached by a vertical access shaft to be commissioned by the Board. The power station is situated in a remote glen, 10 miles from Cannich, Inverness-shire, and is designed for unattended operation.

The entire Affric Scheme was described in detail in the May/July 1952 issues of WATER POWER, but a brief outline of the scheme may be of interest. Loch Mullardoch has been dammed at its eastern end to form a reservoir of 170,000 acre-ft. capacity with a spillweir crest at 817 ft. above sea level. From Loch Mullardoch a 15 ft. 9 in. equivalent diameter partially lined tunnel, 17,102 ft. long, has been driven. The outfall of this tunnel discharges into Loch Benevean, which has also been dammed to give a spillweir crest elevation of 735 ft. From Benevean dam a further tunnel and penstock lead to the Fasnakyle generating station. This is the main installation of the scheme and contains three 22 MW units operating under a net head of 502 ft.

The water level in Loch Mullardoch varies from 725 ft. to 817 ft. and that in Loch Benevean from 715 ft. to 735 ft. There is therefore a gross difference in level of up to 102 ft. between the two lochs. As a

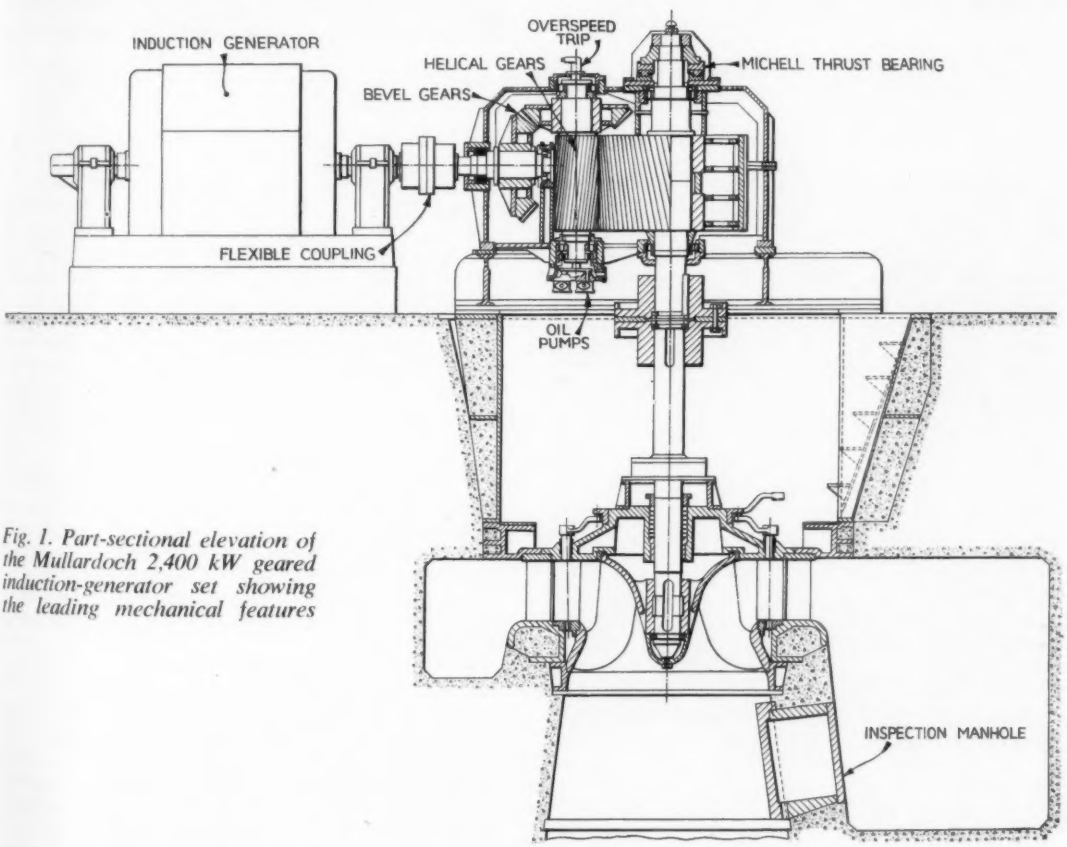

Fig. 1. Part-sectional elevation of the Mullardoch 2,400 kW geared induction-generator set showing the leading mechanical features

flow of over 500 cusecs was intended to pass through the connecting tunnel between the two lochs, the Board decided to install a hydro set to harness the power available. It is estimated that this unit will generate 8 million kWh per annum.

The Civil Engineering Aspect

The Mullardoch plant is housed in an underground chamber having a floor level at an elevation of 744 ft., 80 ft. below the surface. The chamber was excavated from solid rock and its finished dimensions are 34 ft. 4 in. long, 24 ft. 6 in. wide and 25 ft. high. The walls are lined with brick and the roof with asbestos tiles, a cavity being left behind both so that seepage water drains outside the walls and into the main station sump.

Considerable care had to be exercised in excavating the underground chamber as the main Mullardoch to Benevean tunnel had already been constructed. This tunnel was under water pressure and is only 80 ft. from the chamber so that the use of explosives was severely limited.

The machine chamber is reached by a vertical concrete-lined access shaft 12 ft. in diameter. The shaft contains a 4-person passenger lift, a duct for the exhaust air and a cable duct. A segmental space about 12 ft. by 8 ft. is left open for raising and lowering heavy parts of the plant for maintenance. At the top of the shaft is a reinforced concrete structure housing a 7½ tons electric block on a monorail joist. A semi-underground building adjacent to this contains the winch for the inlet gate and the station battery for the emergency d.c. supply.

Water is supplied to the turbine through a 10 ft. 3 in. equivalent diameter tunnel 309 ft. long leading from Loch Mullardoch. During the constructional work the entrance to this tunnel was blocked by a temporary concrete bulkhead. The water from the inlet tunnel passes through a venturi section and then through the inlet gate. It subsequently passes into the turbine spiral which is of reinforced concrete, designed for a static head of 90 ft. of water. The spiral is of rectangular cross-section to simplify construction, the inlet section being 8 ft. square.

The water is delivered from the turbine draft tube into a short 14 ft. 6 in. equivalent diameter discharge tunnel leading back into the main tunnel.

In the main tunnel between Loch Mullardoch and the junction with the turbine discharge tunnel is a 10 ft. by 11 ft. control gate protected by a free-roller emergency gate. This control gate is used to discharge water down the main tunnel when the turbine is not operating.

Turbine Inlet Gate

The inlet gate has a clear opening of 8 ft. by 8 ft. to suit the mouth of the turbine spiral. The gate is of the fixed-roller type with natural rubber downstream seals. Both the roller tracks and sealing faces are of rolled steel with stainless steel cladding on the working faces. The main rollers are mounted on taper roller bearings and the side guide rollers have plastic bushes impregnated with colloidal graphite.

An electrically operated gate hoist is situated at ground level. It is capable of giving a maximum lift of 60 tons at creeping speed to crack open the gate. The hoist is powered by an a.c. motor driving an oil pump which supplies a variable speed hydraulic motor. This in turn drives the rope drums through helical and spur gearing. The gate lifting speed can be varied from zero to a maximum of 4 ft. per minute.

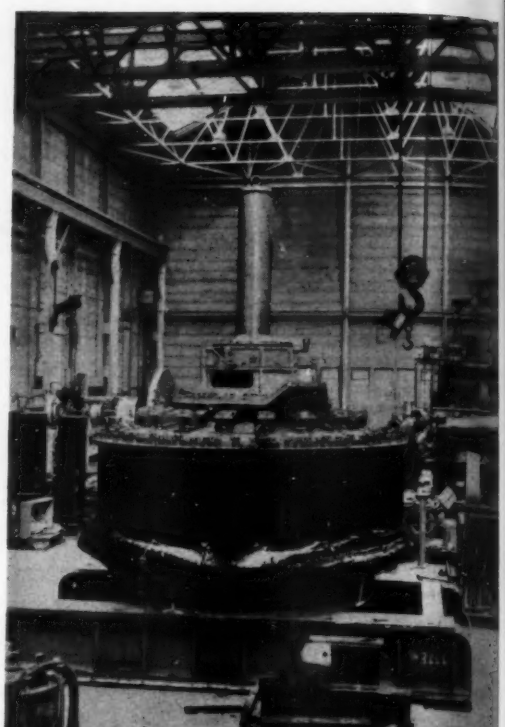

Fig. 2. Turbine speed ring with guide-vane assembly and shaft at maker's works

The speed of gate closure is controlled by the variable-speed hydraulic motor, driven in reverse and acting as a brake pump during the lowering operation. Regulation of speed is obtained by throttling the oil discharge from the motor through cam-operated valves. The speed of descent through the first 70 ft. is approximately 13 ft. per minute and through the last 14 ft. is 1½ ft. per minute to avoid pressure surges in the tunnel. As the gate is lowered under gravity only, all the hoist bearings are of the ball or roller type to reduce friction to a minimum.

The hoisting of the gate is controlled from ground level only, but the lowering operation can be initiated in four ways:—

1. Automatically on turbine runaway.
2. By push button in the generating chamber.
3. By push button at ground level.
4. By manual release of the electro-mechanical brake at ground level.

Turbine and Gear Unit

The water turbine is of the vertical-shaft Francis type and drives the horizontal shaft induction generator through a speed-increasing gear unit. The turbine is rated at 3,600 h.p. for heads between 75 and 90 ft. and operates at reduced output down to a head of 20 ft. It runs at a normal speed of 208 r.p.m.

The turbine is set in a cast-iron speed ring which is grouted into the concrete spiral casing. A welded steel pit liner connects the top of the speed ring to the gear-unit supporting frame. This feature enables the turbine and gear case to be erected at the turbine maker's works, effecting a considerable saving in erection time at site.

The cast-steel Francis runner is of the high specific speed type and is keyed to a forged steel shaft which passes through a Michell self-contained guide bearing carried on the top cover of the turbine. The turbine shaft is rigidly coupled to the gearbox input shaft and the entire hydraulic and deadweight thrust from the turbine is taken by a Michell thrust bearing mounted on top of the gear case.

Cast-steel guide vanes are employed and these are operated through a link and lever system by a 1½ h.p. d.c. motor. The guide vanes have an operating time of five minutes. This was dictated by the severe hydraulic conditions due to the very long tunnel between the turbine and Loch Benevean. The guide vane operating levers are not keyed to the guide vane spindles as a friction grip is employed. This obviates the necessity for the breaking links used by some manufacturers The friction torque on the guide vane spindles is calculated so that a foreign body jammed between two guide vanes will cause one spindle to slip and avoid damage to the operating gear.

Because of the necessarily long guide vane closing time, the unit runs away in the event of complete load rejection. An overspeed trip on the gear unit starts the guide vane motor in the closing direction but the unit reaches runaway speed rapidly before being brought to rest. The generator, gear unit and turbine are therefore designed to operate at runaway speed for about 15 min. With maximum head, the turbine runaway speed is 590 r.p.m. and the corresponding generator speed 2,150 r.p.m.

The draft tube is of the straight taper type discharging into the tailrace tunnel. Access man-holes are fitted between the machine chamber and the spiral casing and between this and the draft tube. The latter manhole enables the runner to be inspected *in situ*.

The gear unit is of the combined helical and bevel type. The first stage consists of a pair of single helical gears having a face width of 33 inches. These connect the input shaft running at 208 r.p.m. to a vertical intermediate shaft running at 712·4 r.p.m. A pair of bevel gears are arranged to give a horizontal shaft speed of 755 r.p.m. The output shaft drives the generator through a gear type flexible coupling.

The gear unit is contained in a welded steel casing supported on the top of the turbine pit liner by a steel frame. It is fitted throughout with ball and roller bearings with the exception of the input shaft thrust bearing which is of the Michell type.

Forced lubrication is provided by twin gear-type pumps driven from the intermediate gear shaft. These pumps supply oil to all the bearings and also direct oil sprays on to the gear teeth. A standby oil pump, driven from the a.c. supply, is provided and is brought into operation during the starting and shutting down sequences. It also starts up should the pressure fail from the main pump. A subsidary use of the standby pump is for emptying the gear unit sump tank and pumping it up the access shaft. A 2 in. oil pipe leads from this pump to the access shaft building and may be used for filling or emptying the gear unit oil sump. This saves the need of lowering oil drums into the machine chamber.

The oil system is fitted with thermostats and pressure relays which are incorporated in the station protective system. The oil sump tank is equipped with immersion heaters to raise the oil temperature to 60°F. before starting. The protective system is arranged so that it is not possible to start the turbine until the oil is at 60°F. and pressure is available from the standby pump. It is also arranged to shut down the turbine in the event of oil pressure failure or excess temperature.

The turbine and gear unit are illustrated in section in Fig. 1 while Figs. 2 and 3 show the turbine in course of construction at the manufacturers' works.

The Induction Generator

The induction generator is a 2,400 kW, 3,300 V machine designed for a synchronous speed of 750 revs. per min. It is of horizontal shaft design with two pedestal bearings. The generator cooling air is drawn from the machine chamber and is discharged into an air duct leading to the top of the access shaft. The squirrel cage rotor is of robust construction to withstand the high runaway speed (2,150 revs. per min.). The rotor core-supporting spider and the shaft are machined from a steel forging and the rotor carries two axial-flow cooling fans. A small permanent magnet generator is connected to the free-shaft end of the machine and supplies both a stroboscopic synchronising device and an electric tachometer.

Low surface temperature heaters are mounted in the bottom of the stator frame to prevent condensation when the unit is not in operation. The stator winding is provided with six embedded temperature detectors.

The electrical control gear is mounted in a three-

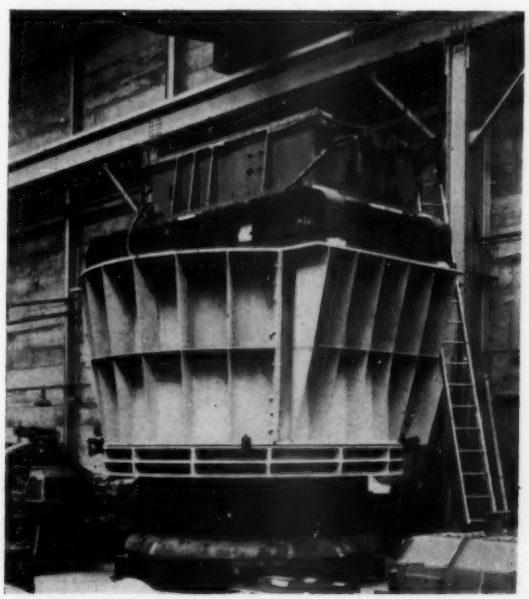

Fig. 3. Speed ring, pit liner and bottom half of gear-case for 3,600 h.p. vertical-shaft Francis turbine

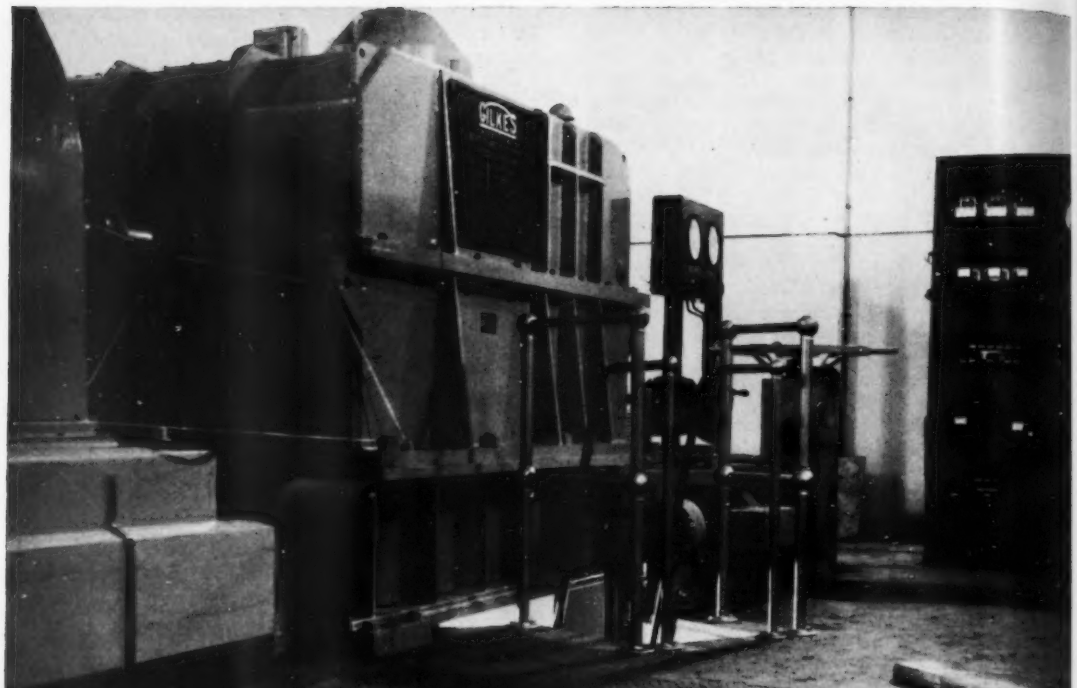

Fig. 4. Interior of generating chamber showing gear unit and turbine access stairway

panel cubicle-type switchboard. This board comprises a relay panel, a control panel and a 3,300 V 75 MVA 3-phase circuit-breaker panel. Auxiliary electrical equipment is supplied from a separate cubicle-type distribution board.

The station is designed for unattended operation although the starting-up, synchronising and shutting-down sequences are at present carried out under local control. Provision has been made, however, for remote control to be effected from Fasnakyle generating station in the future.

In addition to the normal electrical protection, the unit is arranged to shut down automatically in the event of runaway, excess bearing, stator or oil temperature and low oil pressure. A 110 V storage battery is installed in the access shaft building. This battery supplies current for the protection system, the guide vane control motor, the standby drainage pump and emergency lighting.

Station Auxiliaries

All leakage water from the turbine glands and the seepage behind the station walls is carried to a single central sump. This is normally emptied by an a.c. motor-driven float-controlled pump. A duplicate d.c. pump is installed as a standby. A larger a.c. pump is used for dewatering the turbine spiral. All these pumps are of the GGG self-priming water-ring type thus obviating priming trouble and the need for foot valves. The drainage pumps are arranged to deliver leakage water either into the spiral case or the draft tube.

The station is served by a 15 ton hand crane with a 1 ton auxiliary hoist. In addition there is a 7½ ton electric hoist, previously referred to, for lowering equipment down the access shaft. The electric shaft lift is used for carrying personnel and tools.

Conclusion

The author wishes to acknowledge the assistance of the North of Scotland Hydro-Electric Board for their permission to publish the illustrations and for information regarding their plant and also the help afforded by several of the firms referred to below.

The consulting engineers for the civil side of the scheme were Sir William Halcrow & Partners. The civil engineering contractors were John Cochrane & Sons Ltd., and Reed and Mallik Ltd. The main contractors for the generating plant were Gilbert Gilkes & Gordon Ltd., Kendal.

Principal manufacturers were as follows:—

Water turbine and pumps	—Gilbert Gilkes & Gordon Ltd., Kendal.
Gear unit	—David Brown Industries Ltd., Huddersfield.
Inlet gate	—Sir William Arrol & Co. Ltd., Glasgow.
Induction generator	—Bruce Peebles & Co. Ltd. Edinburgh.
Switchgear	—Switchgear & Cowans Ltd., Manchester.
Station battery	—Chloride Batteries Ltd., Clifton Junction, Nr. Manchester.

Vibrations in Hydro-Electric Installations

By SERGE LELIAVSKY, Ph.D., M.I.C.E., M.Am.Soc.C.E.

PART TWO

THE foregoing information on the vibration frequencies of various types of hydraulic equipment —as collected and produced by Mr. S. Judd—may supply the basic design data for the structural project of a power house.

We have seen from Fig. 6 that in order to be safe against excessive vibration, the structure must be designed in such a manner that its natural frequency is either forty per cent. less or forty per cent. more than the frequency of the imposed vibration. In combination with Mr. Judd's data this requirement constitutes the fundamental criterion which we need in order to assess correctly the safety factor against undesirable vibrations.

We can, then, proceed as follows: calculate the power house for static loads alone, and then, find the natural frequency for the structure designed. As explained earlier, this frequency is capable of being determined from the elastic deformation due to the static loads. Now, from the figures given under headings 1 to 5, it follows that the enforced vibrations of the hydraulic equipment and machinery in the power house have frequency ranges of from 1·5 to 150 cycles per sec. (an unbalanced rotor as a minimum, and the turbine runner vibrations in the Keswick power plant, as a maximum).

It therefore follows that the power house must be so proportioned that its calculated natural frequency should be either forty per cent. below or forty per cent. above the specified range.

This requirement yields a clearly defined quantitative parameter for design purposes, but we can go even further than that; in fact, since natural frequency periods of structures of current heights and stiffnesses are seldom longer than the upper limit of the prohibited range, it is obvious that our aim will usually be to produce a design with a natural vibration period *not exceeding* a certain given maximum. In terms of structural design, this means that the calculated deflection (which, as shown earlier, determines the length of this period) must be less than a fixed limit, i.e. in order to be safe against excessive vibration, the power house must possess a certain minimum rigidity, as calculated from the quoted frequencies.

Intuition, blended with a certain amount of commonsense, tells us that a stiff structure resists better the effect of enforced vibrations than a flexible tall building, but neither intuition nor commonsense can furnish the designer with a quantitative assessment of the required minimum rigidity. It is this last information that the figures mentioned under 1 to 5—as interpreted in the light of the elastic vibration theory —are intended to supply.

A few additional explanations may possibly be desirable, in order to understand the intended meaning of such terms as "rigidity" in formulating a code of design rules to ensure safety against vibration.

Let us consider the simplest possible case—a rectangular beam under its own weight. Suppose we find that its natural vibration period is too long and that its static deflection must be, consequently, reduced. Since the span of the beam is supposed to be fixed by extraneous considerations, the dimensions which should be changed are either the width or the height of the section. If we increase the width, both weight and moment of inertia will augment in the same proportion, and since the former appears in the numerator, and the latter in the denominator of the formula for the deflection, it follows that the natural vibration period will not change, and the money spent on the additional material will be wasted.

On the other hand, if it were the height of the section which had been increased, a definite amelioration would be achieved because for a constant width the moment of inertia increases as the cube of the height, whereas the weight is proportional to the first power of the height only. And further, since the natural vibration period is proportional to the square root of the static deflection, it follows that the length of this period is inversely proportional to the depth of the beam.

To simplify the example we have chosen a rather unusual case: a beam which carries its own weight only. In practice, beams are designed to carry additional loads, and in this case, the simple proportionality relations, as stated above, do not hold any more, and therefore, the widening of the beam would not have been altogether useless, as the natural vibration period would thus be reduced.

But, in any case, the effect of a heightening would be far greater.

Suppose now, that for some reason or other, we cannot, or do not want to, alter the section of the beam. What should then be done? The reply is this: if the section must not be changed, there remains one thing only—to provide additional rigidity by altering the methods of fixation. For instance, if the original design was for a simply supported beam, make the beam continuous or fixed at the ends (or end).

This is the solution most commonly adopted in designing power houses, for it is usual to provide in such cases a reinforced-concrete skeleton, made up of a number of continuous elastic frames with rigid corners. In such a composite framework, the natural frequency period for an element does not depend on the rigidity of this element alone, but on all the other members of the system as well. The problem is therefore by no means so simple as in the examples thus far considered. In addition to which it must also be realised that, apart from the individual periods of natural vibrations for every member, the building vibrates as a whole. There may be, also, local foci of vibrations, embracing a certain number of members, which may require to be investigated separately in so

far as their natural frequencies are concerned. Little wonder, therefore, that the solution of the vibration problem from the structural engineering standpoint has lately attracted much attention, and that some publications on the subject have recently been produced.* The fact deserves being emphasised because, according to the earlier, traditional views, the vibration theory belonged to the scope of mechanical engineering, particularly in connection with the whirling of shafts.

It may not be out of place to say a few words at this stage of the discussion on the different "modes" of the vibration. This term is supposed to describe the type of the instantaneous shape of the vibrating beam. In fact, should a parallel be drawn between this shape and that of a wavy sea surface, we may classify the various diagrammatic images thus obtained, either as representing one single wave (see Fig. 11a), or two waves (see Fig. 11b), or three waves (Fig. 11c), and so on.

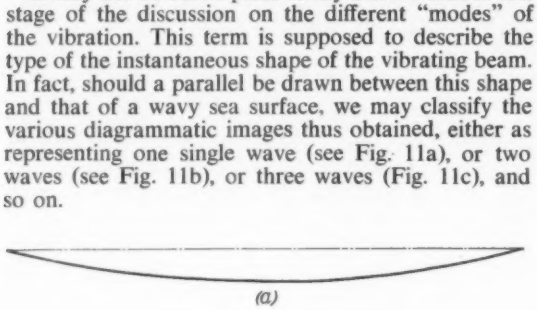

Fig. 11

The first diagram is characteristic of the first, or fundamental mode, which corresponds to the lowest frequency. In the second and third modes, as represented in the *b* and *c* diagrams, or in still higher modes, the length of the wave gradually diminishes, and the period of vibration, consequently, shortens.

Since in this particular investigation we were chiefly concerned with the maximum permissible length of the natural vibration period, the fundamental mode alone was a consideration. In fact, the existence of higher orders of modes—those with shorter periods—did not affect the conclusions derived from our results. But, since we are now considering groups of beams, interconnected in such a way that their natural frequencies are interrelated, we may be faced with conditions in which the modes of the correlated vibrations of the different members of the system will not necessarily be the same. The modes are thus becoming a significant factor of this analysis.

In order to give an idea of the modern solutions of the relevant mathematical problems, the recently published calculation method by Messrs. A. S. Veletsos and N. M. Newmark* may be quoted as an illustrative example. We shall start this quotation with some general principles bearing on the substance of the problems in hand; in the first place, it will be observed that the elementary version of the vibration concept was presented in the foregoing pages in terms of forces and linear displacements, but this presentation may now be generalised, for the same concept can also be expressed in terms of moments and angular deflections—i.e. angles of inclination of the constituent members (beams, etc.) of the analysed system, as measured in degrees or radians. Here again we shall detect the existence of natural vibration frequencies, with the difference only that the sectional areas appearing in the relevant equations will now be replaced by second moments.

Imagine a beam, *ABC* (Fig. 12), which is rigidly fixed at the left end and freely supported at the right end. Originally, the beam is supposed to have been straight, as shown by *ABC* in the drawing; but it is

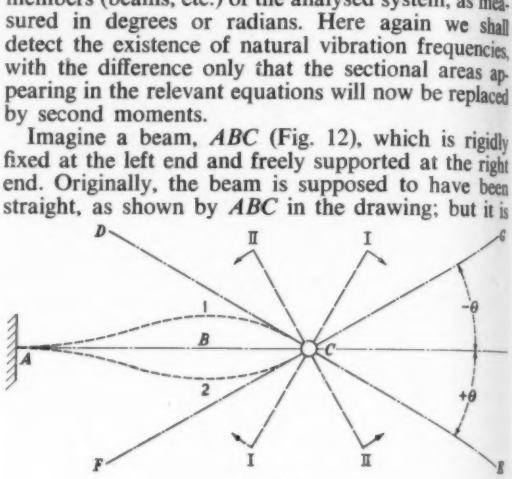

Fig. 12

now in a steady state of forced vibration, due to the effect of a periodically acting couple of forces, i.e., the bending moment, M_C, which is applied in the section over the right bearing.

The extreme values of this variable couple are diagrammatically represented in the figure by the arrows I-I and II-II. The extreme deflected positions of the elastic axis of the beam—corresponding respectively to the extreme values of the couple—are shown by the dotted curves *A1C* and *A2C*. As a measure of deflection we shall take the angle which the tangent to the curve at the point *C* makes with the horizontal axis. Thus, in the extreme positions of the beam (tangents *DE, FG*) the corresponding values of this angle will be $+\theta$ and $-\theta$ (see figure), which are described as the amplitude.

In the ranges in between these extreme values, both moment and angular deflection are harmonic functions of the time; hence, the equations transcribing the vibratory motion of the beam, will be written as follow:

Exciting couple (external moment) $M(t)=M \cos \omega t$
Angular rotation (angle of inclination) $\theta(t)=\theta \cos \omega t$

in which

M is the amplitude of the exciting couple,
θ is the amplitude of the angular rotation,
ω is the circular frequency of vibration,
t is the time.

In order to schematise the presentation we have assumed that the angle of inclination, $\theta(t)$, was measured in the same section in which we have applied

* Cf. A. S. Veletsos, "A Method for Calculating the Natural Frequencies of Continuous Beams, Frames and Certain Types of Plates." Thesis, Univ. of Illinois, 1953. W. Mudrak, "Bestimmung der Eigenschwingungszahlen von durchlaufenden Trägern und Rahmen." *Zeitscher. Angew. Math. u. Mech.*, 1948, p. 258, and many others.

* "Natural Frequencies of Continuous Flexural Members." Proceedings Am. Soc. C.E., Paper 735, July, 1955.

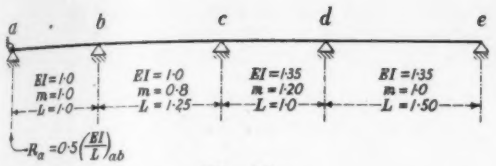

Fig. 13

the external couple, $M(t)$, but this was rather in the way of a special case. In fact, since the entire beam is subject to vibrations, the choice of the point where the vibrations are measured is entirely arbitrary. For instance, if we have a continuous beam, *abcde* (Fig. 13), we can assume that the exciting couple is acting in *e*, that is, in the right-hand end of the beam, and then, measure the angular rotation in any point of the beam, for instance, at the supports or in any other section of the extreme left span.

In this connection it will be observed that in the diagram the main characteristics of this span are taken to be equal to unity and the same constants, for all other spans, are expressed in terms thereof. This method of formulating the conditions of the problem is fully admissible, because the frequency calculations do not depend on the absolute values of the constants, but on their ratios only. We have thus far assumed that in the case of a natural frequency, a finite force, or a finite exciting couple, is capable of producing an infinitely large vibration. But the inverse postulate is also true; in fact, from a theoretical standpoint, in the limit, an infinitely small (i.e. zero) force or couple, can produce finite vibration, provided its frequency is equal to the natural frequency of the beam.

This result is used as a basis for the following calculation: (a) we assume that the beam oscillates in a steady state with an assumed, fixed angular amplitude of rotation at a certain section I; (b) we determine by calculation the magnitude of the exciting couple (external moment), at joint *e* (Fig. 13), necessary to maintain the vibration as assumed under (*a*); (c) we plot the magnitudes of these calculated couples, in a diagram, as functions of the assumed frequencies.

The frequencies, found on such a diagram, for which the exciting couple vanishes, represent, then, the natural frequencies for the beam investigated.

Fig. 14 is a reproduction of such a diagram, as calculated by Messrs. Veletsos and Newmark (*loc. cit.*) for the beam shown earlier in Fig. 13. It will be observed that within the range investigated in this chart, the curve shown therein intercepts the zero line in five points, which all correspond to the desired natural frequencies, but for different modes of vibrations.

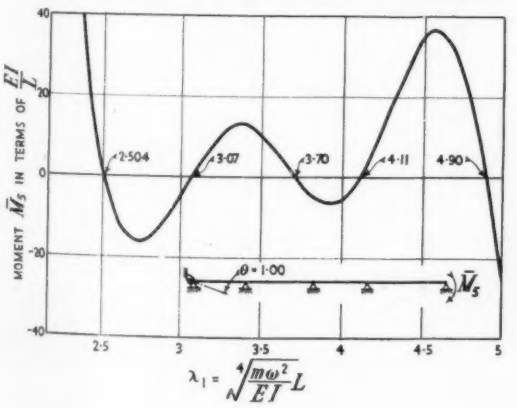

Fig. 14

This last figure completed the presentation of the philosophy of the solution. It remains, however, to show how this philosophy can be applied in the case of a practical problem. Various procedures can, of course, be employed to that end. The solution used by the above authors, is, in a way, an extension of Prof. Hardy Cross's well-known method for continuous beams* and Mr. Gaskell's calculation for vibrating frames.†

We begin by introducing two new constants. Referring again to Fig. 12, we define as *dynamic flexural stiffness* the amplitude of the exciting bending moment applied at *C*, necessary to produce in this same point a steady-state angular rotation amplitude of unit magnitude, and we describe as *dynamic flexural carry-over factor* the ratio of the bending moment developed at the point *A*, to the exciting bending moment at *C*. We shall use the symbols *K* and *k* to denote these concepts, which, it will be observed, are descriptive and characteristic of the structural and elastic properties of the beam intended to be investigated.

Armed with these two new tools the designer can carry out the required calculation step by step, starting with the span in which he assumes a certain oscillation, and finishing with the joint where the external exciting couple is supposed to be applied. In fact, since our main equations are linear and the principle of super-position is valid, it can easily be shown that for consecutive spans in a continuous beam

$$(M_n)_R = K_n\theta_n + (kK)_n\theta_{n+1}$$

and $$(M_n)_L = K_{n-1}\theta_n + (kK)_{n-1}\theta_{n-1}$$

in which $(M_n)_R$ and $(M_n)_L$ are the bending moments immediately to the left and immediately to the right of the *n*th support, while all other symbols represent rotations and bending moments for the spans indicated by the indices (viz., $n-1$, n and $n+1$).

Also, since axiomatically

$$(M_n)_L + (M_n)_R = 0,$$

it follows that

$$\theta_{n+1} = -\frac{(kK)_{n-1}\theta_{n-1} + \bar{K}_n\theta_n}{(kK)_n}$$

in which $\bar{K} = K_{n-1} + K_n$. (Note that k does not appear in these equations separately, but is always multiplied by K, which explains the use of a single index for the product of these two factors).

These equations, strictly speaking, are valid for interior supports only, but analogous expressions can easily be developed for the end supports also. In this case, however, the form of the equation will depend on the stiffness, R, of the elastic restraint at the end. Thus, the relationships between moments and rotations for the first and last supports in Fig. 12 will be, respectively, $M_a = -R_a\theta_a$ and $M_e = -R_e\theta_e$. For a hinged end $R = 0$, and for a fixed end $R =$ infinity. Negative signs in these formulae are used to show that the restraint acts always in a direction opposite to the direction of the movement.

---

* "Analysis of Continuous Frames by Distributing Fixed-End Moments," Trans. Am. Soc. C.E., Vol. 96, 1932, p. 1.

† "On Moment Balancing in Structural Dynamics," Quarterly of Appl. Math., Vol. I, 1943, p. 237.

With these notations the above formula for θ reduces to the following equations:—

left end $\quad \overline{K}_a\theta_a + (kK)_a\theta_b = 0$

where $\overline{K}_a = R_a + K_a$

right end $\quad \overline{M}_e = (kK)_d\theta_d + \overline{K}_e\theta_e$

where $K_e = K_d + R_e$

The last equation applies to the case of a hinged (or partially fixed) end. For a fixed end $\theta_e = 0$, and the criterion for a natural frequency is then that this equation be satisfied.

It should be clear from the foregoing text that provided the value of K and kK are known, our problem is practically solved. These values have been tabulated by the same authors* in terms of the factor EI/L, depending on the parameter

$$\lambda = \sqrt[4]{\frac{m\omega^2}{EI}}\,L$$

in which:—

m is the mass per unit of length of the beam,
ω is the circular frequency of vibration,
E is the modulus of elasticity,
I is the second moment,
L is the span.†

The tables in question are computed for values of λ between zero and 10·20, to seven significant figures, at increments of 0·01. This relatively high precision is in this case a practical necessity to obtain significant results, but in order to give an idea only of the general behaviour of the functions here investigated, the same authors produce the diagram represented in Fig. 15, in which both K and k are plotted against the corresponding values of λ. For more information on this interesting subject the reader is referred to the original publication.

Similar, but rather more involved, methods are used if instead of a continuous girder, the designed structure is a continuous elastic frame with rigid corners —as will most often be the case with modern power houses. Here we may have to consider two different types; in fact, the frame may either be "open" or "closed." In the former case it consists only of beams, or of slabs which are perpendicular to the plane of vibration. In the second case, the slabs are parallel to this plane.

It goes without saying that the estimate of the natural frequencies for the second type calls for still more elaborate calculation methods than those which will suffice for the first, more common type.

One point is fairly obvious, viz., that computations such as those described in the foregoing, require both time and trained staff, and since, as explained earlier, the design of the structural skeleton of a power house against vibrations is, of necessity, a trial-and-error process, steps should be taken to reduce to the greatest possible extent the number of repetitive calculations of the natural frequencies of the structure. Then, should the first original project be designed with respect to static loads alone, material alterations may subsequently be required to make the structure strong enough to withstand the effect of the vibrations, and this may, very possibly, mean repeating the frequency calculations many times all over. To reduce (or avoid) such repetitions, various attempts have been made to use empirical rules, which may help to form, from the start, a general idea about the importance of the vibrations to be expected in each particular case of power house design.

* Messrs. Veletsos and Newmark, see footnote page 188.

† The last three symbols have the same meaning as before, and are repeated here for the convenience of the reader only.

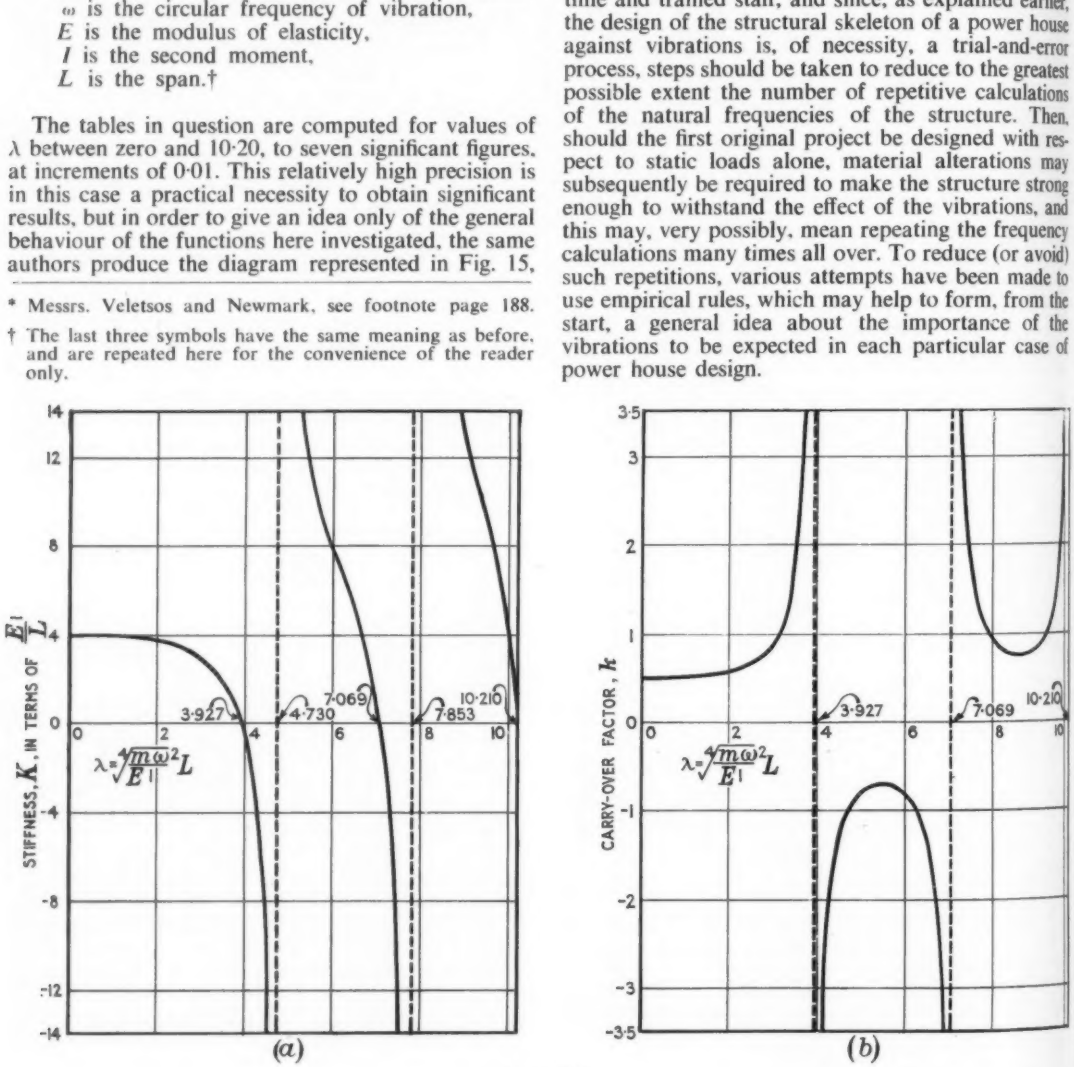

Fig. 15

As is usual in establishing empirical formulae, the main problem was to find the true parameters, i.e. the basic variables to appear in the relevant criteria. Since the early thirties, the Bureau of Reclamation* were using for this purpose the minimum concrete mass in the structure, on the one hand, and the kilowatt capacity of the installation on the other. While the empirical rules obtained in this manner are admittedly a very rough guide only, the argument is that certain losses are inherent in transforming the flow of water into electrical energy through mechanical means.

The energy lost in this transfer is approximately 15 per cent. of the capacity of the hydro-electric plant. Though what part (probably small) of this amount is attributable to vibration, may be questionable, it is nevertheless believed that this part must be absorbed so as not to become objectionable—under the wide range of impressed frequencies and forces. To do this, the Bureau use, as an empirical design criterion, a minimum value of 2 cu. ft. of the concrete mass surrounding the scroll case and supporting the generator for 1 kW capacity of the machine. It should be realised, however, that in existing installations this ratio is by no means constant—not even approximately so. In fact, in a well-known list published by Creager and Justin, this ratio varies from 1 to 30.

Studies tending to establish an empirical correlation between the mass in question and various characteristics of the plant installed, such as the revolutions per min., the head, the discharge, the specific speed and the horse power, were undertaken by the Bureau, but led to no reliable results.

It is believed, at present, that a more satisfactory empirical design rule might be to adopt a limit for the maximum acceleration due to any periodic force. For instance, values ranging from 0·005 g and 0·01 g have been suggested, in which g is the acceleration of the force of gravity.

Other parameters have also been attempted, e.g., some practical designers tried to establish an empirical proportion between the weight of the rotating parts and those of their supports respectively. Thus, in certain cases, the generator supports and scroll encasement concrete were proportioned to provide a minimum weight of two to two-and-a-half times the weight of the rotating parts.

None, however, of these sporadic attempts has, thus far, been generally accepted or recognised. Their design values remain, therefore, a matter of personal opinion.

A somewhat different approach has been made by Mr. Mallock*, who has studied vibrations from the standpoint of the noise they produce. According to this author a periodic vibration becomes perceptible to the ear when the acceleration attains one per cent. of gravity. This criterion for "nuisance vibration" is:—

$$4\pi^2 x_o F^2 = 0{\cdot}05\ g$$

where x_o is the displacement,
F is the frequency,
g is the acceleration of the force of gravity.

This method has not yet been applied to the design of power houses in hydro-electric plants.

Another aspect of the vibration theory of power houses, which deserves possibly being more thoroughly explained, is the damping concept. We have thus far considered viscous damping only, because its mechani-

* The following information is based on Samuel Judd *ut supra.*

* See Dr. S. E. Slocum, "Noise and Vibration Engineering"; quoted from S. Judd *ut supra.*

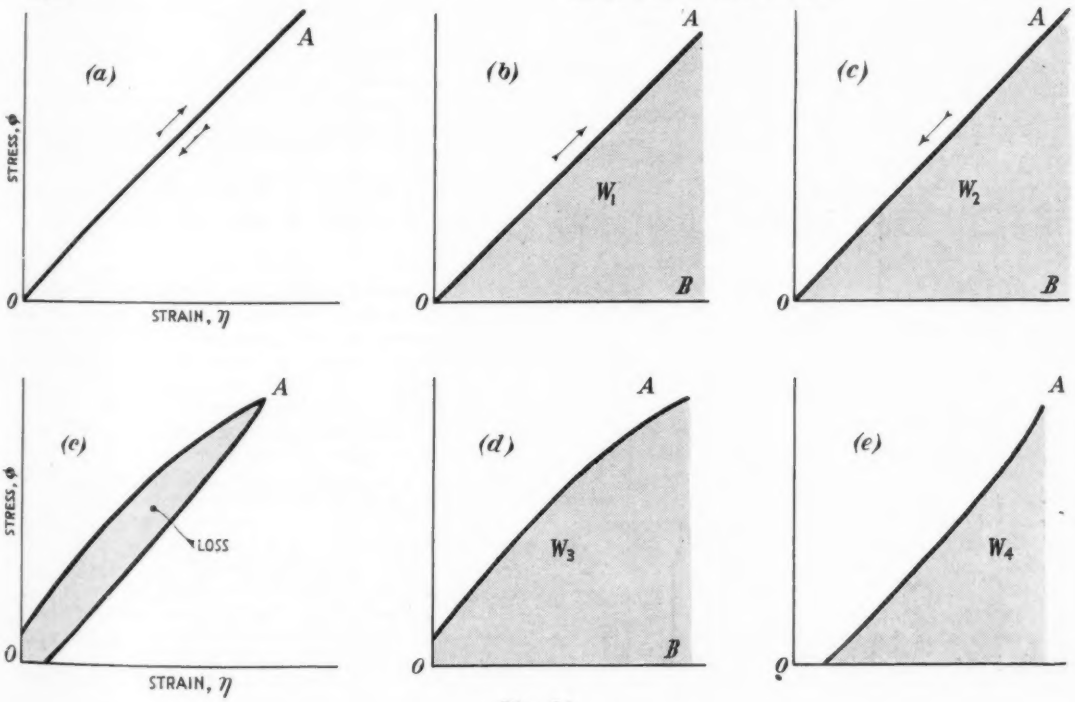

Fig. 16

cal model was the most obvious and the easiest to explain, but it is not typical of the vibrations of buildings such as power houses.

In this case, by far the more important factor is the "internal friction" or "hysteresis damping." The term hysteresis is well known, but its application to vibration may call for a more explicit formulation. To start with, let us consider the common type of stress-strain diagram as shown in Fig. 16(a). It consists really of two superposed curves, corresponding respectively to the rising period, as in Fig. 16(b), and to the falling period, as in Fig. 16(c). The elastic energy, stored as stress energy during the first period, is represented by the area W_1, of the triangle OAB in Fig. 16(b), whilst the energy given up during the second period is equivalent to the area W_2 in Fig. 16(c). Since the curves for the first and second period fall upon one another, W_1 is equal to W_2, i.e., the stored energy is the same as the energy given up, and the balance of the operation is, consequently, zero.

Such assumptions are, however, fully illusory, their object being chiefly to help the beginner to understand the principle of modern stress-theory. What occurs in nature, is that whatever branch of engineering we may be contemplating, no accumulator is ever perfect, and the energy given up must therefore be less than the energy stored.

In our case, since the material is not ideally elastic, the stress required to produce a certain strain during the rising period—as shown in Fig. 16(d)—is always slightly (may be, very slightly) higher than the corresponding stress in the falling period—as in Fig. 16(e). It follows that the curve in (d) is located higher than in (e), so that when these diagrams are superposed, as in (c), the respective curves form a loop, the enclosed area representing the "loss" of energy, partly converted to heat.

The foregoing is a well-known explanation, bearing on the stress-strain diagram *in general*. If now, we consider the case of vibration *in particular*, we must add to the diagram Fig. 16(a) another branch, as shown in Fig. 17. For instance, let us suppose that the movement is from the left to the right. Then as this weight, or the beam, or whichever structural member we are investigating, passes through its neutral position, corresponding in the diagram to the origin O, the velocity is a maximum whilst the stresses are zero,

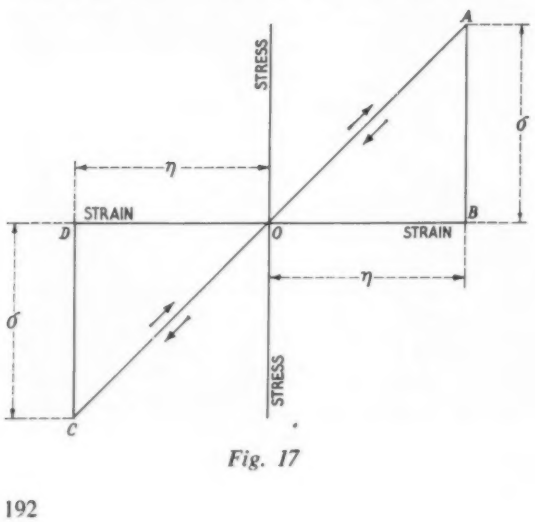

Fig. 17

which means that the energy is dynamic energy alone, while potential energy is zero. (For simplicity we assume horizontal motion only, so that "energy by virtue of position" does not interfere with the main argument.)

As the movement continues, the velocity gradually drops and the stress rises. From energy standpoint this means that dynamic energy is constantly being transformed into an equal amount of potential energy of stress. This will continue until the point A is attained, when all the dynamic energy is expanded and there is only potential stress energy.

After that an inverse process will begin, i.e., the potential energy will drop and the dynamic energy will rise, until, at point O, all energy is again dynamic.

The left hand branch of the diagram is interpreted in precisely the same manner, and we have thus a complete graphical model of the process of energy transformation in a vibrating ideally-elastic member.

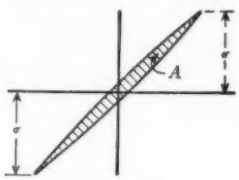

Fig. 18

Here again, however, the effect of imperfect elasticity will create conditions similar to those represented earlier in Fig. 16(d); in fact, as shown in Fig. 18, which is abstracted from a paper by Messrs. F. Bleich and L. W. Teller,* the two branches of the curve form a loop (hysteresis) the enclosed area (hatched and designated in this diagram by the symbol A) giving quantitatively an estimate of the loss of energy, i.e., the intensity of the "internal" damping.

These authors have carried out a number of interesting tests on the subject, their argument being:—"Damping capacity of a structure may be defined as the ratio of the energy ΔW dissipated on one cycle of oscillation to the maximum amount of energy W accumulated in the structure in that cycle."

The damping capacity—a non-dimensional quantity—is therefore expressed mathematically

$$\psi = \frac{\Delta W}{W}$$

It will now be shown that though ψ is not ideally constant, it may be assumed as such, in an approximation suitable for systematising experimental figures.

In Fig. 19 (which is abstracted from the same source) let W be the stress energy accumulated at time t, and let W_2 represent the energy accumulated at time $t_1 + T$ in which T is the period of one oscillation.

* "Structural Damping in Suspension Bridges," Proc. Am. Soc. C.E., Separate No. 61, March 1951, p. 3.

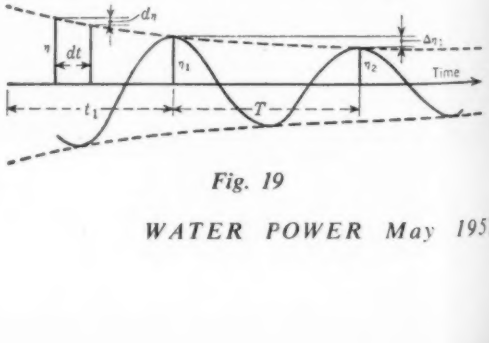

Fig. 19

It is obvious that owing to the effect of damping $W_2 < W_1$, and:—

$$W_1 - W_2 = \Delta W \text{ and } \psi = \frac{W_1 - W_2}{W_1}$$

By assumption the motion is essentially a small motion, i.e. the amplitude is of a lesser order of magnitude than the overall dimensions of the vibrating structure. Also, although the elasticity of the material is not ideal, we may, nevertheless, assume for an approximate estimate that in Figs. 18 and 19 the amplitude η is proportional to the stress σ, and therefore, the area of the triangle representing the energy W is proportional to the square of η, hence, $W = k\eta^2$. It follows that

$$\psi = \frac{k\eta_1^2 - k\eta_2^2}{k\eta_1^2} = 1 - \frac{(\eta_1 - \Delta\eta_1)^2}{\eta_1^2} = 2\frac{\Delta\eta_1}{\eta_1},$$

neglecting $\frac{\Delta\eta_1^2}{\eta_1^2}$ as a quantity of the second order.

For the sake of this discussion let us now assume that ψ is a constant. (The proof of the validity of this assumption is left till a later stage of this discussion, depending on the results of the tests themselves.)

If the above formula is, then, applied to the infinitely small part of the curve corresponding to the time interval dt (see Fig. 19), the differential equation will be found to be

$$-\frac{d\eta}{\eta} = \frac{\psi}{2}\frac{dt}{T}$$

$$\text{Integrating: } log_e\, \eta = -\frac{\psi t}{2T} + C$$

$$\text{Hence, } \eta = e^{C - \psi t/(2T)} = \eta_o e^{-\psi t/(2T)}$$

in which η_o is the original amplitude of the vibration at the beginning the decaying process, i.e., at time $t = 0$.

Considering, now, two consecutive maxima, η_r and η_{r+1}, at times t and $t+T$, we find

$$\frac{\eta_{r+1}}{\eta_r} = \frac{e^{-\psi(t_r+T)/(2T)}}{e^{-\psi t_r/(2T)}} = e^{-\psi/2}$$

which gives the rate of decay of the amplitude.

The dimensionless factor $\delta = \psi$ is referred to in this investigation as the "logarithmic decrement of damping," because

$$log\, \frac{\eta_{r+1}}{\eta_r} = -\delta$$

Experimental results published by the authors of this analysis tend to show that although δ does not remain constant for all cases, it varies only slightly within certain specific ranges. For instance for steel, within the limits of dynamic stress between 1,000 lb. per sq. in. and 3,000 lb. per sq. in., δ may be assumed to have an average value of 0·004.

It is believed, in general, that the value of δ for concrete is many times that for steel. For instance, for steel stacks lined with concrete, Messrs. Dockstader, Swiger and Ireland* estimate the value of δ at 0·08, working from data obtained by Muckenhoupt.†

In concluding this brief review of modern tendencies in connection with the problem of hydro-electric power house vibration, it must be realised that considerable progress towards its solution has been achieved in recent years. The question that still deserves to be clarified is the individual vibration of single members or groups of members which may occur independently from, or in parallel with, the vibration of the power house as a whole. For example, instances were reported in which part of the power house was in resonance with disturbance caused by a compressor, or a sump pump. Such conditions are almost impossible to take into account when the designs are being prepared and means must therefore be adopted to carry out corrections after the plant has been completed and put into operation.

* "Resonant Vibration of Steel Stack," Proc. Am. Soc. C.E., Separate No. 541, November, 1954.

† "Study of Concrete as a Material for Turbine Foundations," Unpublished Report.

Italian Consultants for Japanese project

Dott. Ing. Carlo Semenza of Electroconsult has announced the signing of an agreement between his company (a corporation recently established by three Italian industrial groups: FIAT, Montecatini and SADE) and the Osaka Kansai Electric Power Company Inc. of Japan. According to this agreement, Electroconsult will co-operate with Kansai's technical department in planning and designing a giant dam to be built in the Japanese Alps. This dam, thanks to the highly-perfected building techniques to be employed, will be an outstanding engineering project; it has been given the name "Kurobe IV." It is intended to control the flow of the river Kurobe by setting up a reservoir with a capacity of about 200 million cu. m. The size of the dam, which will be dome-shaped, places it among the most important projects of this kind. The height will be about 190 m. and length of the crest about 380 m., while the volume of concrete used will be about 1½ million cu. m. An underground station will be equipped with three generating units, giving a total capacity of about 270,000 kW.

A great deal of development work has been carried out in Italy on the technique, design and construction of dome-shaped dams. They are specially adaptable to exploit the static properties of materials in the most efficient way and, as proved by analysis and experience, the safest in the event of exceptional stresses. As the Japanese archipelago is, on the whole, subject to frequent and widespread earthquakes, the possibility of seismic stresses have to be borne in mind. The final design will be checked, before construction begins, by a series of tests on a model, which will be carried out in the specialised laboratories of ISMES at Bergamo.

Protection of Iron and Steel

Corrosion Limited, Southampton, claim that their Glopane Universal paint, which is a zinc-based material, can be applied equally well on iron and steel in wet, damp, or dry weather, and can even be applied under water if necessary. They state that after brushing on to wet steel, this paint will dry hard enough in about 24 hours to withstand abrasion, leaving a tough coat of zinc that renders any rust inert, and protects the structure from rusting any further. Coverage is approximately 24 sq. ft. per lb. to give an average one-coat application cost of 1½d. per sq. ft.

A battery of ten Atlas Copco AR 4 Compressors installed on the Snowy Mountains Project, Australia.

Atlas Copco AR type Compressors give *more* air to the h.p. than other machines of their class

Choose an Atlas Copco AR Compressor and rely on it to deliver more air, h.p. for h.p., than any other compressor of its class. These machines have been developed with the aim of low fuel and running costs. Take the AR 3 featured in the table below. Its power consumption at 100 lbs. per sq. in. is 105 h.p. The supply of free air is 570 cu. ft. per min! To get the same amount of air from other compressors would require an additional 15 h.p., but 15 h.p. never used mean an annual saving on fuel bills of £270. That amount represents approximately one-fifth of the initial cost of the AR 3. In five years, that's your money back!

Flexible Air Supply

Three-step valve unloading control on smaller models or five-step clearance pocket control on larger sizes permits flexible adaptation to variations in air demand.

Low Discharge Temperatures

Amply-dimensioned water pockets, the two-stage design and streamlined air passages give discharge temperatures below 270°F at 100 lbs. per sq. in.—compared with 475°F from single-stage machines. Lower temperatures mean longer valve life and elimination of air receiver and pipe-line explosions.

Extra-Solid Construction

High-class Swedish steel throughout; amply-dimensioned parts reduce stresses to a minimum; crankshafts carried on SKF roller bearings; cross-head design with ground or white-metal lined bearings eliminates wear.

Ease of Installation

Right-angled, double-acting cylinders combined with counterweighted crankshaft provide a low weight/capacity ratio and good balancing, thus reducing foundation requirements. So good indeed, that most models can be mounted on a skid underframe and used as semi-portable units without any foundation or bolting down.

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AR 1	120	600	400	330	62	310	3000
AR 3	120	500	715	570	105	500	5400
AR 4	120	429	910	760	138	680	6600
AR 5	120	375	1235	1000	186	880	9000
AR 7	120	333	2120	1740	318	1520	14500
AR 9	120	300	4020	3210	586	2920	26400

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Book Reviews

Veröffentlichunggen zur Erforschung der Druckstossprobleme in Wasserkraftanlagen und Rohrleitungen (Investigations of water-hammer problems in water-power plants and pipes), edited by Dr.-Ing. habil. Friedrich Tölke, docteur ès-Sciences h.c., professor at the Stuttgart Technical University. Published by Springer-Verlag, Berlin/Göttingen/Heidelberg, 1956. 7½ in. by 10½ in., V-149 pp., 225 ff. Price in Germany, paper wrappered DM.37·50.

This is the second issue of a series of highly valuable contributions to the study of water-hammer problems. It consists of five papers by distinguished German experts, the first by Professor Tölke himself, dealing with the design of those sections of a water-power plant which may be endangered by water hammer. Following an introductory chapter on generalities, the author examines the preventive measures to be taken against penstock bursting, proceeding from a correct selection of the quality and thickness of the steel plates. Chapter III deals with the possibilities of utilisation of prestressed concrete in pressure conduits, and chapter IV surveys the various constructional aspects of the various types of surface and underground hydro-plants in relation to the layout of the power tunnels and penstocks. The final paragraph of this chapter contains short considerations on the comparative behaviour in the occurrence of water hammer in tunnels and shafts on the one hand, and penstocks on the other. This paper concludes with an investigation of the dependence of water hammer on the law of closure.

The second paper is a report by W. Pelikan, Stuttgart, on control measurments at an Alpine power station on a penstock laid in an inclined shaft with a view to ascertaining the reinforcing effect of the concrete casing on the penstock. The factors which contribute to a reduction of the strengthening effect of the concrete casing are reviewed, and the description of the tests is followed by the results of static elongation measurements when filling and emptying the penstock.

Paper No. 3, by Dr.-Ing. H. Blind, Munich, investigates non-permanent flows in various types of tailrace tunnels, separate chapters being devoted to free-flow tunnels and the surge phenomena occurring in them, tailrace tunnels on the borderline between gravity and pressure conduits, tailrace tunnels under pressure, and tailrace tunnels operating without a surge tank. The last two papers, contributed by W. Gandenberger, Stuttgart, and Dr.-Ing. H. Stack, Hannover, respectively, deal with water hammer such as occurs in domestic supply pipes, and the calculation of the size of air chambers in waterworks.

Stavba Přehardy, Komplexnì Mechanisace Betonářských Praci (Dam Building, Systematic Mechanisation of Concreting Work), by Gustav Pavlík and Viktor Chandži, and associates. Published by Státnì Nakaldatelství Technické Literatury (State Publishing Office for Technical Literature), Spálena 51, Prague II. 7 in. by 9½ in., 340 pp., 22 ff. Price in Czechoslovakia, linen bound, Kčs. 37·80.

As far as we are aware, this is the first work devoted especially to the mechanisation of concreting work in its connection with dam building. That it should be written by Czechoslovak technicians for the benefit of their colleagues in the project offices and in the field appears less surprising when one learns that there are at present no fewer than 16 concrete dams in course of construction in their country, not to mention a number of developments still at the preparatory stage. The book is divided into 29 chapters which deal in condensed form but quite comprehensively with all the problems arising in a systematic application of mechanisation to concreting work in dams, and with all the machinery and equipment involved. No aspect of any of these problems has been forgotten, beginning with the quarrying of sand and aggregate material and ending with the placing of concrete and its treatment after pouring. No less than 54 pages are devoted to concrete technology, and the importance of this chapter is accentuated by the fact that it was written by Professor Stanislav Bechyně, author of a standard work on the subject. Handling the transport at all stages is paid special attention, and there is even a short chapter with valuable time-saving suggestions on the transport of workers to and from the construction sites. Another important section of the book we should like to single out is chapter XXIV on the electrification of dam construction plants. The final chapter deals with additives and the latest improvements in the manufacturing of special dam cements. It is interesting to note that, instead of references in the text of diagrams to existing examples of practical applications, the only structure instanced is an imaginary gravity dam set up in imaginary surroundings: it would therefore be a waste of time to look on a large scale map for the Odrava and Lubina Rivers and the townships of Luby and Kotirov whose names appear throughout the book, as none of them exist. Dams and surroundings, however, have been so judiciously selected that the fictitious map inserted at the beginning of the book will prove as useful as any actual layout, and the various solutions suggested can easily be applied to a practically unlimited range of sites and structures. The illustrations, mostly line, are excellent and show that Czechoslovak dam concreting and construction plants are fully up to date; looking at them will make the English would-be reader feel doubly sorry that the language difficulty proves such an obstacle.

Canadian Model Studies

As part of a water power development in northern Quebec, the Mechanical Engineering Division of the National Research Council of Canada has constructed a model of a chute to be used for transporting logs over the dam. It is built to a scale of 1:25, which results in a model 20 ft. long. The chute will carry 500 cusecs, and tests are being conducted to evaluate its hydraulic characteristics.

The two models of reaches of the St. Lawrence River continue to be used for investigations on both the navigational and the power phases of the Seaway project, and a 1:30 scale model has been built of one of the locks at Soulanges to assist in the design of the filling and emptying systems which will operate the lock as quickly as possible commensurate with safe stresses in the ships' hawsers.

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Scottish Hydro Board Report

THE report of the North of Scotland Hydro-Electric Board for the year ended December 31, 1956, has much of interest to say about the steady development of hydro-electric capacity in the Scottish Highlands, and reveals, perhaps unconsciously, the enlightened view that the Board take of their task in bringing prosperity to the country they serve. During the year 101·5 MW of new hydro-electric capacity was brought into commercial operation, and it is estimated that a further 85 MW should be ready to commission by the end of 1957. Altogether, 660 MW is in operation, 210 MW under construction, 38 MW promoted and awaiting construction, and 704 MW in course of promotion and survey. The eventual capacity envisaged is upwards of 1,612 MW, providing an average annual output of 3,917 million kWh.

The Board's hydro-electric, steam and diesel stations generated 1,744 million kWh during the year, an increase of 312 million units compared with 1955, due partly to the wetter year and partly to the additional plant brought into service. The new plant was available when the rainfall was high at the end of the year and helped to raise the output from hydro-electric plant alone to the new record figure of 11 million kWh in 24 hours. The year's hydro production of 1,282 million units was greater by 52½% than that for 1955.

The constructional scheme for the River Farrar in Glen Strathfarrar and the River Beauly at Kilmorack was further advanced and brought to the stage of being ready for publication. The scheme to develop the resources of the River Awe and its tributaries, the Nant and the Cruachan Burn, was also brought to a more advanced state of preparation. It possesses features which make it attractive for development for pumped-storage purposes. Investigation of pumped-storage schemes continues because of the flexibility and operating advantages they offer in association with large nuclear and high-efficiency conventional steam stations. In addition to more detailed examination of the development of pumped storage at Loch Awe, site investigations have been carried out at Burn of Mar (Balmaha) and at Loch Sloy, both of which would utilise Loch Lomond. To solve some of the problems that arise there test model investigations have been put in hand.

Investigations into the development of the water-power resources of the River Tay at Kenmore and Grandtully were directed mainly to the fishery problems involved.

The total length of rock tunnel driven during 1956 was 19·25 miles or 8,500 ft. a month. The total now driven is 129 miles and including the contracts let during the year there are still 28 miles to cut. Further experimental lengths of tunnel were lined with Gunite to obtain experience of its suitability as a protection for poor rock. Similarly the Aliva process, where a heavier and denser concrete is sprayed on to the rock surface, has also been applied in one or two places. The protective lining of a small diversion tunnel with bricks in the side walls and precast concrete ribs for the roof is also being tried out.

The dams at Quoich, Invergarry and Cluanie, where preliminary impounding started towards the end of 1955, were completed during the year and before the end of 1956 were in full use up to their ultimate capacity. Further work on the dams at Lawers and Errochty permitted the partial impounding levels to be raised by stages. Early in 1957 both these dams will be certified to store water to the final level. In December the dam in Glen Shira reached the stage when partial impounding could be started. The storage here will be of considerable advantage in operating the Clachan station. The Allt-na-Lairige prestressed concrete dam was completed in September and the reservoir was full when the power station was started up in December. At the end of the year the dams at Glascarnoch, Vaich, Fannich, Lednock and Shin were all approaching completion, while at Lubreoch approxmiately 72% of the total concrete had been placed.

Fly ash continued to be used in the concrete at Lubreoch and Lednock dams and increased the total saving of cement to approximately 5,500 tons. At Cluanie and Loyne dams, where wet-ground blast-furnace slag was used, the total saving of cement amounted to 36,220 tons. Further savings amounting to 10,190 tons were made by using blast-furnace slag in the linings of the tunnels at Upper Glenmoriston and St. Fillans.

The Errochty power station, which came into use with one set in 1955, reached its full capacity of 75 MW when early in 1956 two more sets were completed and commissioned. The 20 MW high-head Kaplan-type turbine at Invergarry came into service about the same time. Much later in the year the Ceannacroc station in Upper Glen Moriston came into full operation when one set of 16 MW and one of 4 MW were commissioned.

The somewhat smaller stations at Kilmelfort, Achanalt, and Allt-na-Lairige were commissioned in the second half of the year and in December a third 950 kW set was added in Storr Lochs power station. Allta-na-Lairige power station has one 6,000 kW horizontal-shaft generator driven by a double-runner Pelton turbine, and will be remotely controlled from the Clachan station about three miles away. The Achanalt power station with a vertical Kaplan-type turbine and a 2,400 kW induction generator will normally be unattended and have its output controlled by the level of its head pond.

Two hundred circuit miles of 132 kV lines were put into commission during the year bringing the total up to 1,480 miles. A further 310 circuit miles are in course of construction. There are now 28 substations in operation on the Highland Grid with a total installed transformer capacity of 665 MVA and a further five are under construction.

Icing of conductors, even in the absence of wind, creates such difficult problems that the Board have installed special switching arrangements to enable relatively heavy current at lower voltages to be fed into such 132 kV lines as are suffering from icing and thus undo or retard the build up of ice. Investigations are

ELECTROMECHANICAL INDUSTRIES

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also being pursued into the best means of obtaining early information about the onset of icing conditions, and experiments with equipment for detecting ice by measuring tension in the conductors also are being made.

Development work has continued on electronic fish counters for the purpose of adopting them for use with the Borland fish lock, and some further experiments have been made to improve the design of electric fish screens. Model experiments on a slotted-weir type of fish pass were carried out for the Board by the Dundee Technical College.

Trials of underwater television as a means of inspecting screens, intakes, etc., were carried out at Loch Mullardoch in conjunction with The Marconi Company. Screens at the main tunnel intake at a depth of about 70 ft. were examined, and considering the improvised arrangements for handling the equipment, very satisfactory results were obtained. The screen bars were clearly depicted on the receiving screen and the nature and extent of debris readily identified. The trials showed that television offers great possibilities for the inspection and maintenance of underwater structures.

The report also surveys the Board's activities in fish conservation, roads, stone building and housing, electrical distribution, and the development of industrial, domestic and rural utilisation of electricity.

Abstracts from the World Technical Press

Remote-Control System at Kotla Power Station

This paper describes and analyses the various features of the remote-control system of the Kotla power station, which was commissioned June 9, 1956. The plant, which is now equipped with two 24,000 kW generating sets, provision being made for a third unit of the same size, is controlled from the Gangewal power station, about six miles away upstream, both plants constituting the lower stage of the Bhakra-Nangal project. This is the first application of automatic remote control in Indian hydro-electric developments. (G. S. Gyany & S. S. Nindra, *Power Engineer*, New Delhi, Vol. 6, No. 4, October 1956, p. 267, 5 pp., 5 ff.)

A Two-Flow Tidal Scheme

Double-flow tidal projects are nothing new; to mention but one such scheme, the plan to harness the Menai Straits to power generation is already some years old. There is however a fundamental difference between this scheme and the project advocated and surveyed in this article. Whereas the former involves the closure at both ends of the natural power channel provided by the Straits, the latter is actually an "inland" plant built on a power canal which, combined with two rivers, cuts across the Cotentin peninsula from coast to coast, both sections of the canal acting alternately as headrace and tailrace. Two schemes are suggested, both based on the Douve estuary on the east coast, and, on the west coast, on the Ollonde and Ay estuaries respectively, the latter or Carentan-Lessay scheme only being dealt with in detail, as topographically more favourable and altogether more economical. The working of the plant is not founded on the fact that average tides on the west coast are 2·5 m. higher and 2·5 m. lower than on the east coast, but on the difference in time between the tides on the two coasts, a difference which can be increased artificially by siting the power house far enough from the east coast; thus, there would be high tide on the west branch of the canal and low tide on the east branch, and vice versa. There are two likely transition sills between the two watersheds: Bellefontaine on a tributary of the Ay, and Le Val, on a tributary of the Séves. Simplicity in design and construction appears to be the outstanding feature of a double-flow scheme of the pattern advocated by the author, since there is neither dam, nor intake, nor sluice spillway, and the power house itself is built on firm ground. There is but one unknown quantity: the velocity of the flow in the power canal, the computation of which would be extremely difficult, but model tests would certainly solve the difficulty. The author advocates a plant with an installed capacity of 120,000 kVA, divided between fifteen 8,000 kVA generating sets of the submerged bulb type, without gates, and with reversible propellers. Such a pilot-plant could reach an output of 50,000 kW and a production of 385 million kWh a year over a working period of 7,700 hours. The layout of the power house and of the power canal, the tidal regime, possible period of utilisation, flow velocity and dimensions of the canal are discussed at length. (L. Kervran, *Le Génie Civil*, Vol. 76, No. 3426/7, August 1956, p. 285, 5 pp., 3 ff.)

California's Three-Dam Project

As indicated by its name, this project centres around three dams now in course of construction on the Stanislaus River: Donnells dam, farthest upstream; Beardley dam, about 12 miles downstream, and Tulloch dam, some 45 miles farther down river. The main purpose of the scheme is irrigation, but there will be a power house at each dam, the aggregate generating capacity being 81,000 kW. Donnells dam is the most important of the three structures insofar as power production is concerned. It will deliver water to a 7·2 mile, 12½ ft. horseshoe tunnel connecting with with the power house by a 2,600 ft. long, 81 in. O.D. penstock, the drop between tunnel portal and power house being 1,200 ft., and the maximum static head on the 54,000 kW generator, 1,481 ft. The major problem at Donnells dam has been excavation of streambed gravels to bedrock, which required a hole 200 ft. deep. The cofferdam protecting the hole is simply a facing of the natural material by a cable-bound timber cribbing. The bottom section of the dam will be merely a concrete plug filling the deep excavation, the concrete arch structure starting at about streambed level. Concrete placing began last July and, should no unforeseen difficulty be encountered, dam

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and power house should be completed and in operation by late this year, a year ahead of schedule. (*Engineering News-Record*, Vol. 157, No. 7, August 16, 1956, p. 40, 6 pp., 8 ff.)

Investigating Air-Flow in Circuit Breakers by the Schlieren Method

The schlieren method has proved to be of value when investigating air-flow conditions in air-blast circuit breakers. With its aid, fluctuations in density of the flowing air can be measured, and the flow lines made visible, direct information being thus obtained on the suitable designing of the air-flow channels and contacts, so that the current of compressed air has an effective extinguishing action on the arc. This article commences with a short account of the schlieren method with special reference to its application to air-flow research in circuit breakers, and then describes the air-flow measuring equipment, which consists essentially of two lens systems between which a circuit breaker element or a scale model is placed. This element is fitted with windows which enable light to pass through the air flow to be investigated. The circuit-breaker or its model connects through a valve with a tank supplied with air under pressure by a compressor. The air pressure in the tank can be varied, and the stream of air through the circuit breaker is released by means of the valve. In the final section of the article, various schlieren photographs of the air flow in air-blast circuit breakers are analysed. (W. Pucher, *ASEA Journal*, Vol. 29, No. 9, 1956, p. 107, 6 pp., 12 ff.)

High-strength Steel Scrollcases

The use of T-I high-strength alloy steel, developed three years ago by U.S. Steel Corporation for bridges, pressure vessels, dredge pipelines and truck bodies, has now extended to turbine casings. First specified early last year for four of the Brownlee dam's turbines, it was subsequently called for in scrollcases at Noxon Rapids dam in Montana and Swift dam in Washington. The agencies involved are obviously satisfied that the high cost of T-I steel and special welding requirements are more than balanced by the greater thickness of the flange-quality carbon steel usually employed for turbine casings. Scrollcase plate thickness, it is claimed, can be reduced by about 50% with T-I steel, making the net material cost run about equal to flange-quality steel. It is understood that designers of several hydro-projects are seriously considering its use in penstocks. (*Engineering News-Record*, Vol. 157, No. 20, November 15, 1956, p. 33, 1 f.)

Bottom-Outlet Discharge Measurements

This article describes and discusses the discharge measurements taken on a ring-seal gate which controls the closure of the bottom outlet of Mooserboden reservoir of the Glockner-Kaprun development. These tests were based on the case of penstocks for which it is possible, from the position of the control valve and taking the static pressure into account, to determine the discharge without using additional measuring implements, once the relation between gate position and discharge has been once and for all ascertained by an absolute measuring method. In the case instanced, the relation between gate position and discharge had been previously determined by means of a Texas-type Ott propeller meter the blades of which had been conveniently protected from damage likely to result from the high flow velocities. Before being used in the tests, the meter had been thoroughly tested at the State Hydraulics Laboratory in Vienna. On the basis of the tests, the discharge through the gate at any of its positions can be computed by means of a simple equation which takes into account the static pressure in front of the gate. (Dr. H. Böhm-Raffay and Dr. K. H. Fasol, *Wasserwirtschaft*, Vol. 47, No. 1, October 1956, p. 9, 4 pp., 4 ff.)

Reinforced-Concrete Research

This is a report on the first two conferences of the "Institut de Recherches Appliquées du Béton Armé" (Institute of applied research in reinforced concrete), held from April 9-18 and from November 9-24, 1956. The following are a few of the wide range of papers read at those two conferences: "Study of Surface Foundations" and "Technology of Concrete, considered as a Raw Material," by R. L'Hermite; "Introduction to the calculation of Breaking Strength and to Research towards an Improved Productivity in Concrete Techniques," by Y. Saillard; "Theory of Electroplastic Flexure," by L. P. Brice; "Organising Construction Sites," by A. Balens-Beau; "Laying out a Concreting Plant," by M. Ducret. (*Travaux*, Vol. 41, No. 268, February 1957, p. 96, 11 pp.)

Montelimar Development

A detailed description is given of the Rochemaure dam which retains the Rhône water and feeds it through a diversion canal to the Chateauneuf-sur-Rhône power station. The author then gives an account of the various phases of construction of the structure, scheduled on the one hand to leave an adequate outlet to the river flow and, on the other, to ensure uninterrupted navigability while work was in progress. Great difficulties were encountered during the floods of December 1954 and January 1955, when the discharge of the river rose to a maximum of 6,300 cu. m. per sec. (January 19). All work had to be stopped; not only did bays 5 and 6 of the weir remain submerged for two months, but erosion in the navigation pass exceeded several metres. (C. Baier, *La Technique Moderne-Construction*, Vol. XI, No. 10, October 1956, p. 255, 6 pp., 10 ff.)

Grout Makes Damaged Spillway Sound

This article describes the procedure applied to repair the stepped spillway of 50 year old New Croton dam, a structure of the New York City's water supply system damaged by a heavy storm in October 1955. Spillway and main dam form a continuous structure, with the spillway at right angles to the dam and connected by a circular curve 250 ft. long. The dam, designed for a flood flow of 8,000 million gallons a day, had to cope with a total run-off from the storm of about 48,000 million gallons, and when the lake was drawn down, cracks in the cut-stone masonry of the upstream face of the spillway came into view, while in the downstream face several stones forming the steps showed vertical cracks. From these cracks and separated mortar joints, water leaked at the rate of 1·5 to 2 million gallons a day. The first stage was to caulk the upstream face with lead wool, a method which

had proved successful on the main dam in 1908. Then core borings were started through the heart of the damaged section of the spillway, 15 vertical holes being drilled from the top step, 8 diagonal from the seventh step, and six vertical and six diagonal from the eleventh step. After all the holes had been completed, grouting proceeded in stages of 20 ft. lifts, the operation being effected by means of 1 in. diameter pipes fitted with expansive rubber plugs to prevent the grout from rising above the level at which it was being placed in a lift. All holes were grouted to the same level before shifting the plugs to the next higher level. Upon full completion of the job, two core holes were drilled from the top step and subjected to water and dye tests. No leaks occurred, thus establishing that the original structural integrity of the spillway had been restored. (*Engineering News-Record*, Vol. 158, No. 5, January 31, 1957, p. 35, 3 pp., 8 ff.)

A Joint Russo-Norwegian Venture

The Pasvik River rises in Lake Inari in Finland at just about the beginning of the Russo-Norwegian border, 25 miles of the river being in Russian territory and 50 miles forming the mutual border. Engineering teams from the two countries concluded preliminary survey work last September and three dams are under consideration for the river stretch common to both countries, two to be owned by U.S.S.R., one by Norway, all three however to be built, according to reliable sources, by Norwegian contractors. Total potential of the three power stations is expected to exceed 100,000 kW with a year-round average of 75,000 kW. (*Engineering News-Record*, Vol. 157, No. 17, October 25, 1956, p. 52, 1 map.)

Reopening a 300 kV Kitimat Line

This article, an adaptation from a paper read by the author before the Engineering Institute of Canada, describes the spectacular procedure applied to restore the 300 kV Kitimat transmission line in the Kildala Pass, where it had been knocked out in late January 1955 by an avalanche which destroyed three towers and halted production of Aluminium Co. of Canada's smelter at Kitimat. The plan ultimately adopted and carried out consisted in suspending the two circuits of 3,364 MCM ACSR conductors to two steel wire ropes strung across the valley over a distance of more than 6,500 ft. Security against any rope failing and fouling the circuits was obtained by yoking the two cross ropes by means of two bridle-plates spaced 400 ft. apart, one on each side of the centre section supporting the conductors. Thus a failure of one rope or one anchorage could drop one of the half sections of rope, but the bridle plate involved would arrest the movement and protect the system. Further, each rope is strong enough to support the entire system with an ice loading of 15 lb. per ft. on all components. The four anchorages being set at different elevations, the rope lengths from bridle plate to anchorage also differ, the four main conductor spans are unequal, and none of the end towers is at the same elevation. Each anchor consists of six 2½ in. diameter deformed anchor bolts that are 25 ft. long, upset, and threaded 15 in. at one end; the bolts are grouted into a pattern of 4½ in. holes dipping at 15° and 45°, and are splayed up to 30° maximum on each side of the centre line. It is a remarkable fact that, although three transmission towers were removed from each line, static loadings are still reduced on each of the end towers. This was achieved because: (1) the cross suspension carries more of the conductor weight than was carried by the six towers it replaced; (2) the more direct route of the new system produced a decrease in line angle at the four end towers. In addition to the preliminary model tests, effected with piano wires and lead weights, the paper describes the difficulties encountered in erecting the ropes and stringing the conductors, re-stringing the re-used left line proving particularly awkward. This cross-suspension system had been in operation for over a year at the date of writing, and has been kept in observation under normally severe conditions of wind and icing. It has behaved in a very docile manner and is now considered a secure and stable system. (H. B. White, M.E.I.C., *Electrical World*, Vol. 146, No. 26, December 24, 1956, p. 82, 4 pp., 6 ff.)

Hydraulic Jump Studies

These short notes deal with the loss of head resulting from a hydraulic jump in a plane flow. The author first considers the case of a flow inside a tunnel with a horizontal floor and a rectangular cross section wide enough to render the influence of the side walls negligible and to enable the flow to be considered as plane. The hydraulic jump separates two zones: upstream a shooting flow with a given velocity U_1 and a depth h_1, perhaps topped in the most general cases by a turbulent zone of a depth equal to the total depth of the flow at that point less h_1, the uppermost space being filled by air at a pressure y_1; downstream, the flow, with a velocity U_2, fills the tunnel, submitting its roof to a pressure y_2. From the continuity equation $q=U_1h_1=U_2h_2$ and an application of the Euler theorem to the flow between the two terminal sections under consideration, the author deduces general formulae which can be applied in their turn to particular cases of special interest, such as: (1) The case in which the upstream shooting flow is free; (2) hydraulic jump of a submerged jet in an open channel; (3) hydraulic jump of a free jet in an open channel. (L. Escande, *Revue Générale de l'Hydrauliaue*, Vol. 21, No. 73, July—August 1956, p. 33, 3 pp., 4 ff.)

CLASSIFIED ADVERTISEMENTS

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